BSE

***B**rienen **S**tructural **E**ngineers, P.S.*

PRFO20240573

THE APPROVED CONSTRUCTION PLANS AND ALL ENGINEERING MUST BE POSTED ON THE JOB AT ALL INSPECTIONS IN A VISIBLE AND READILY ACCESSIBLE LOCATION.
PRINT in COLOR and to SCALE.

Approval of submitted plans is not an approval of omissions or oversight by this office or noncompliance with any applicable regulations of local government. The contractor is responsible for making sure that the building complies with all applicable building codes and regulations of the local government.

City of Puyallup
Building
REVIEWED
FOR
COMPLIANCE
RayC
05/08/2024
11:12:51 AM
CITY OF PUYALLUP
STATE OF WASHINGTON

City of Puyallup
Development & Permitting Services
ISSUED PERMIT

Building	Planning
Engineering	Public Works
Fire	Traffic

Centeris Data Center Shed
1023 39th Avenue South East
Puyallup, WA 98374

Foundation Permit Submittal
Structural Calculations

PAUL J. BRIENEN
STATE OF WASHINGTON
32114
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER

Project Number 24202
04/05/2024

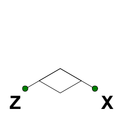
Phone: (206) 397-0000 1316 Central Avenue South, Suite 200, Kent, WA 98032 www.bse-ps.com

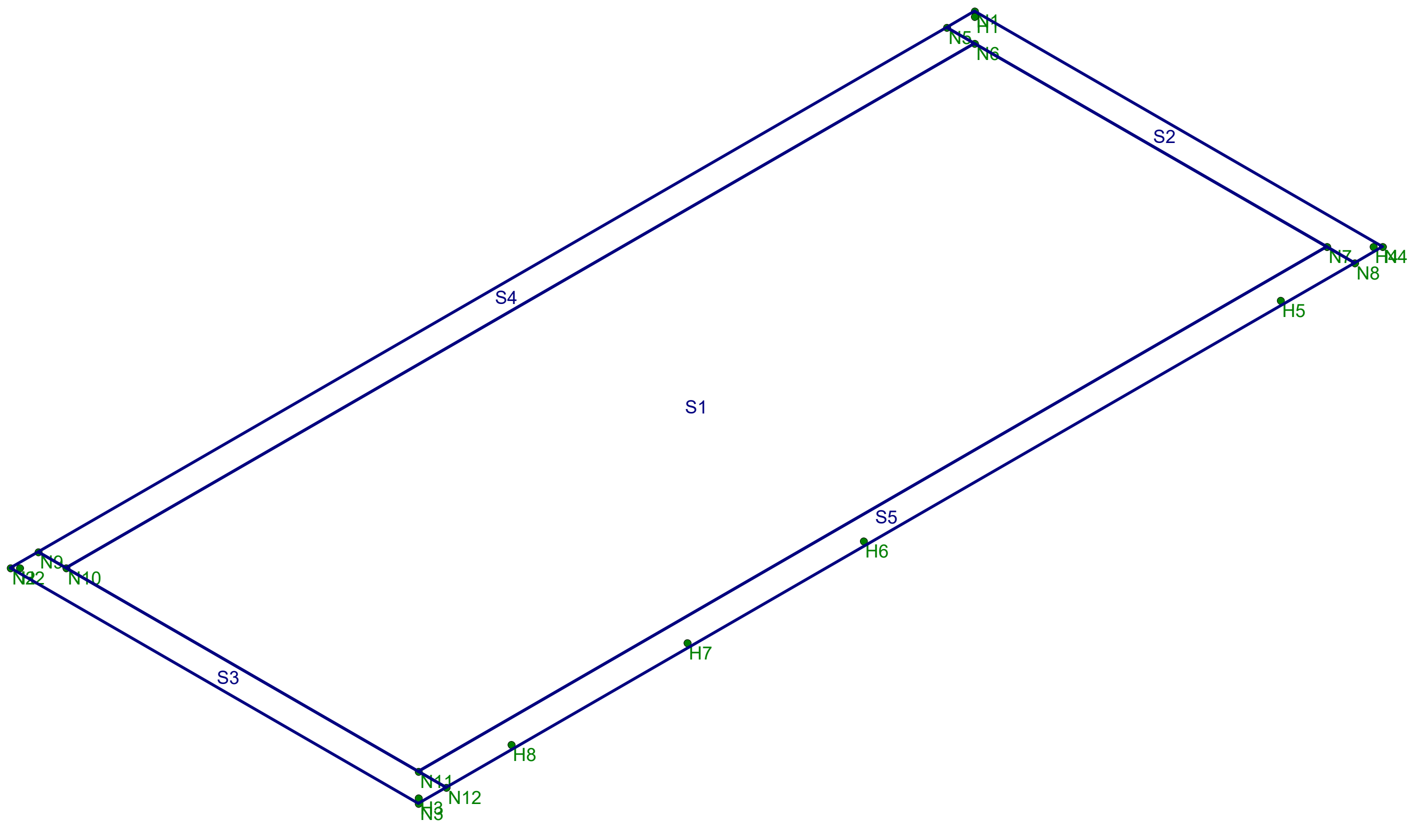
NOTE TO THE REVIEWER:

THE CALCULATION PACKAGE IS BEING SUBMITTED FOR THE FOUNDATION PERMIT SUBMITTAL ONLY. HOWEVER, IN ORDER TO DETERMINE THE LATERAL AND GRAVITY LOADS ON THE FOUNDATION, THE WHOLE STRUCTURE IS INCLUDED IN THIS CALCULATION PACKAGE FOR REFERENCE ONLY. THE SUPERSTRUCTURE WILL BE PERMITED SEPARATELY AT A LATER DATE.

THE FOUNDATION DESIGN OCCURS ON PAGES 3 - 19. THE SUPERSTRUCTURE LATERAL DESIGN OCCURS ON PAGES 20 - 46 (WHICH INCLUDES HOLDDOWN & SHEAR ANCHORAGE), AND THE VERTICAL DESIGN ON PAGES 47 - 63.

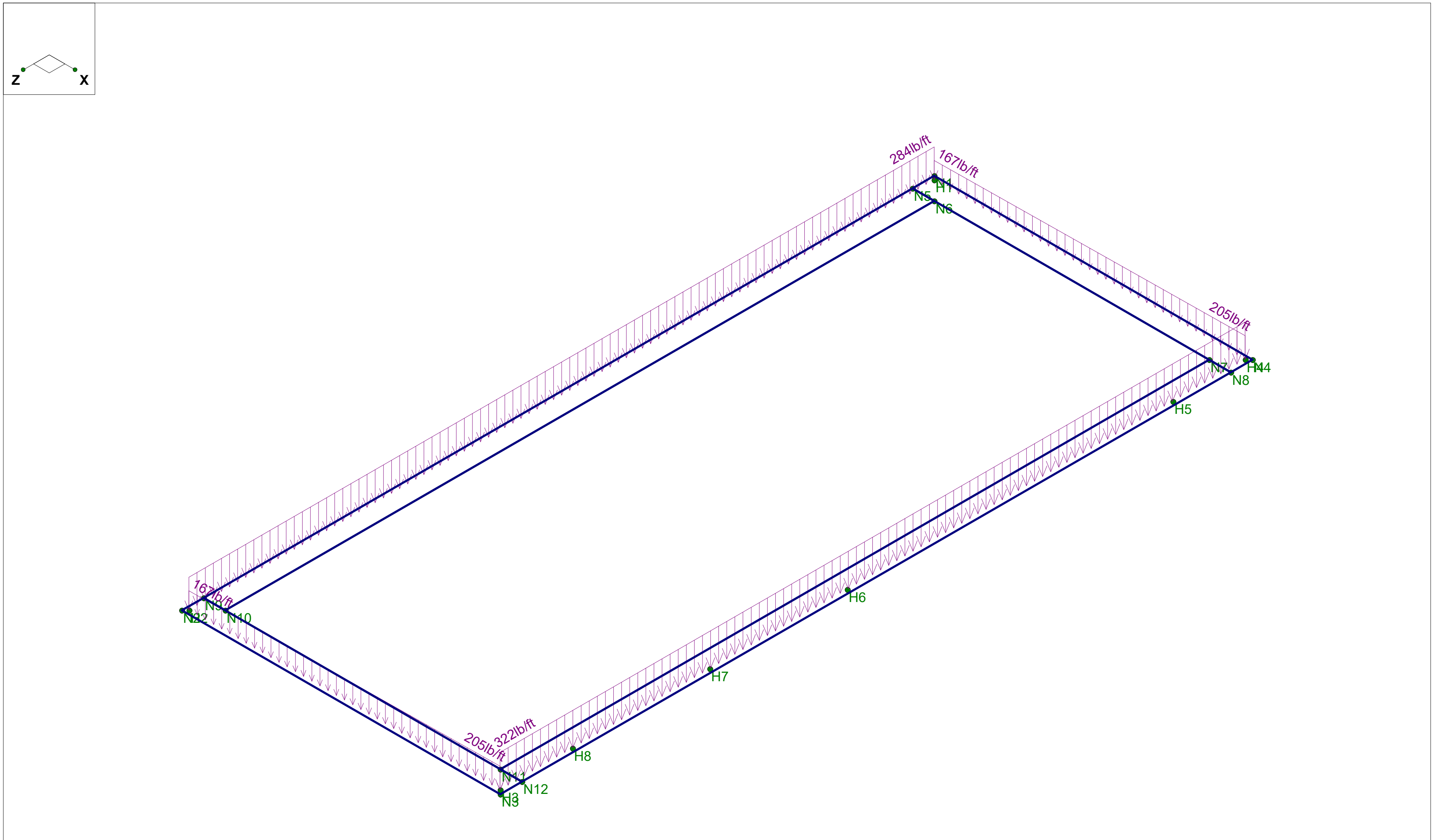
RISA Foundation





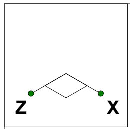
Results for LC 1, ASCE 1

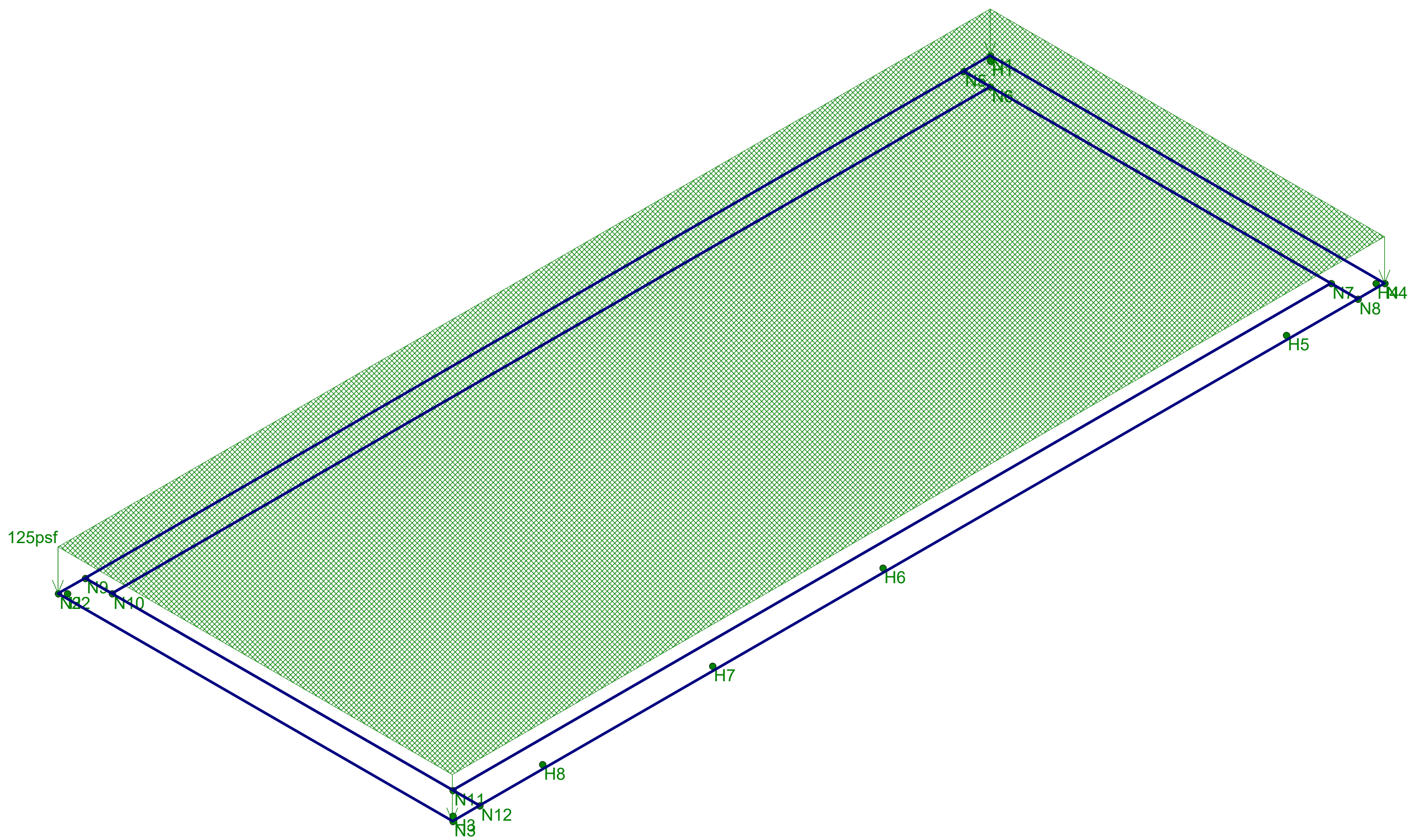
		SK - 1
		Apr 4, 2024 at 2:36 PM
	Slab Labels	Foundation.fnd



Loads: DL - Dead Load
Enveloped Results

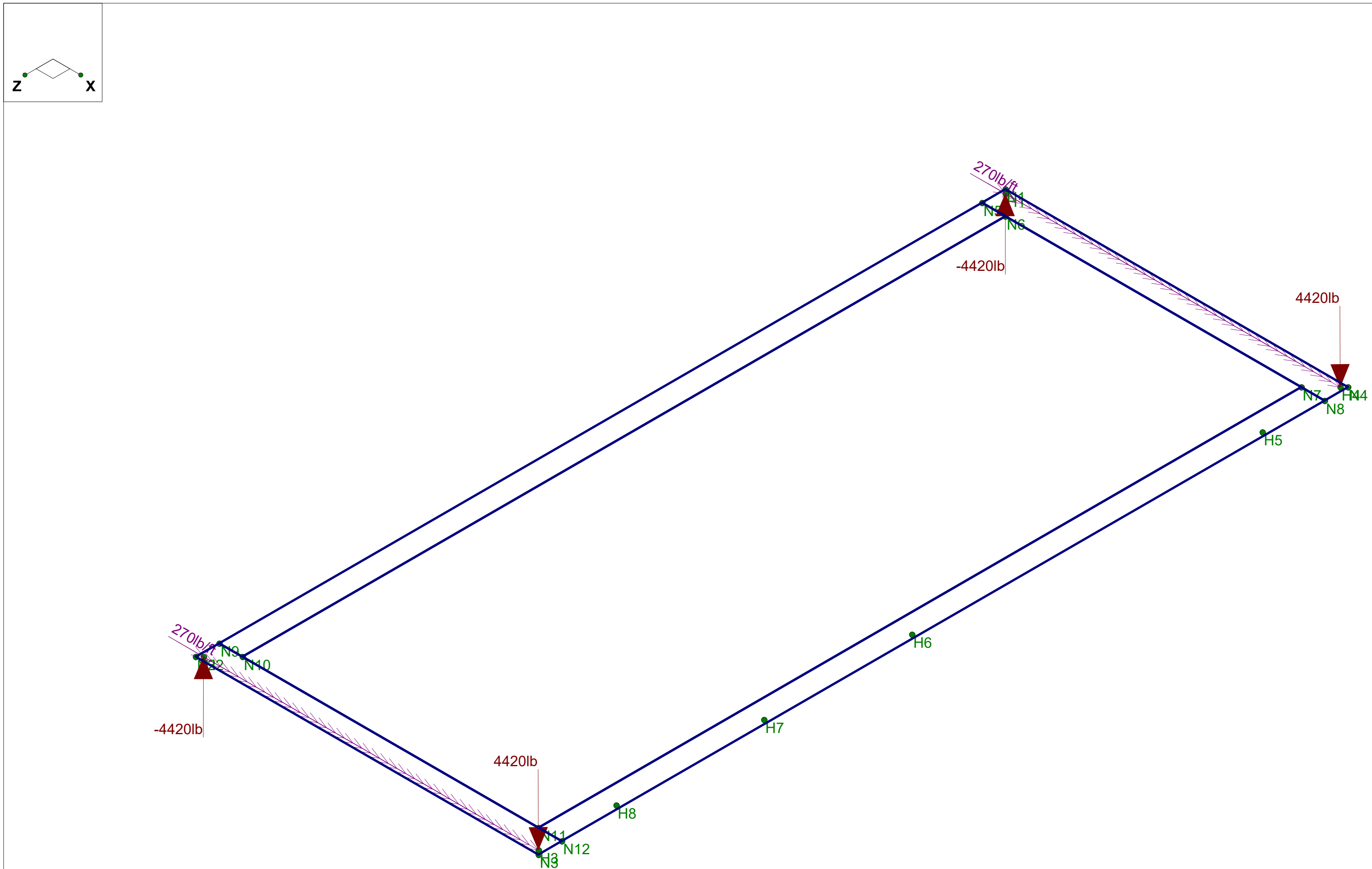
		SK - 7
		Apr 4, 2024 at 3:25 PM
	Dead Load	Foundation.fnd





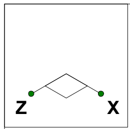
Loads: LL - Live Load
Enveloped Results

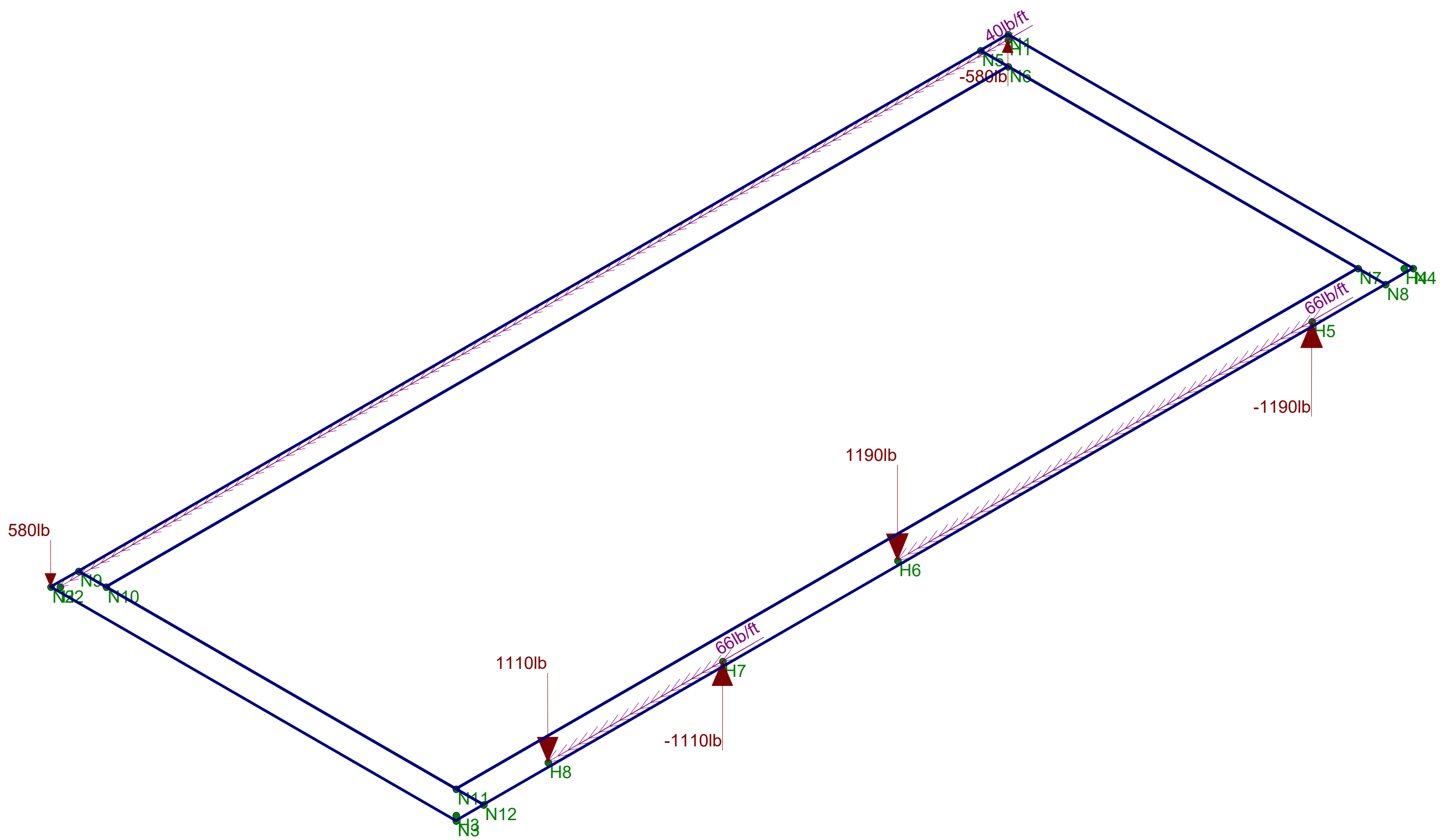
		SK - 9
		Apr 4, 2024 at 3:28 PM
	Live Load	Foundation.fnd



Loads: WLX - Wind Load X
Enveloped Results

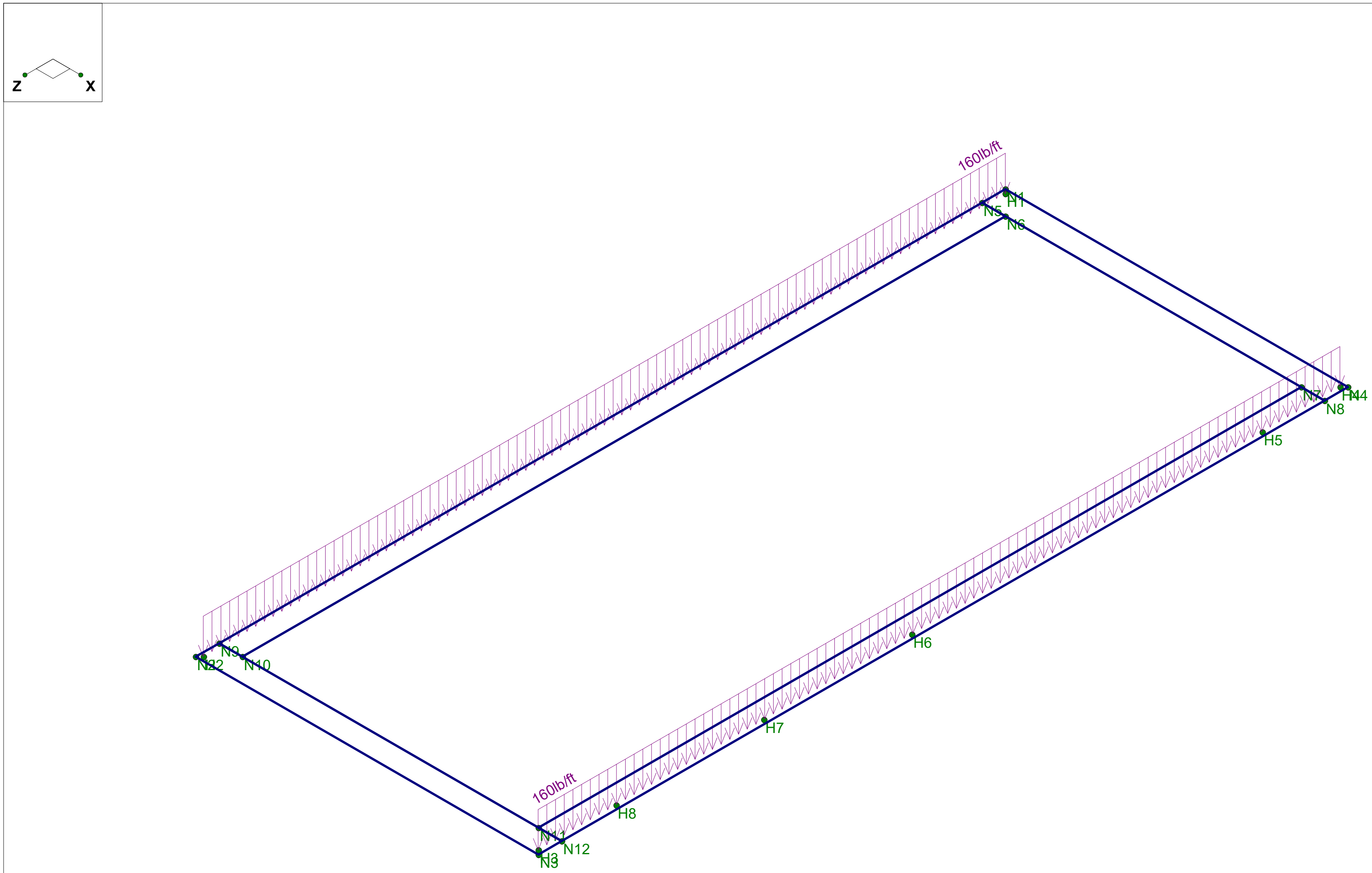
	Wind Load X-Direction	SK - 5
		Apr 4, 2024 at 3:22 PM
		Foundation.fnd





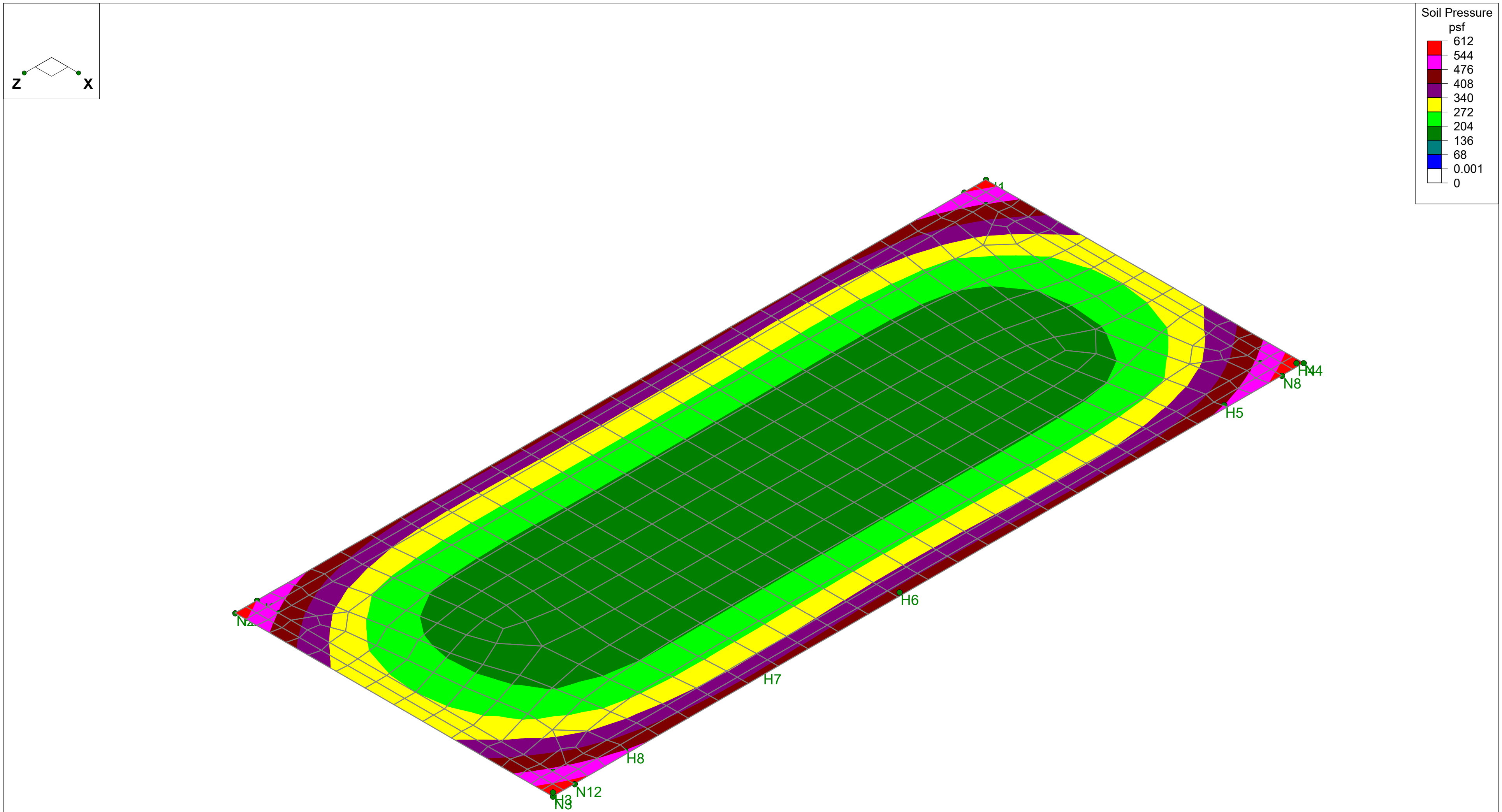
Loads: WLZ - Wind Load Z
Enveloped Results

	Wind Load Z-Direction	SK - 6
		Apr 4, 2024 at 3:23 PM
		Foundation.fnd



Loads: WL - Wind Load
Enveloped Results

	Wind Load Y-Direction	SK - 8
		Apr 4, 2024 at 3:26 PM
		Foundation.fnd



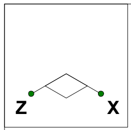
Loads: EPL - Earth Pressure Load
Enveloped Results

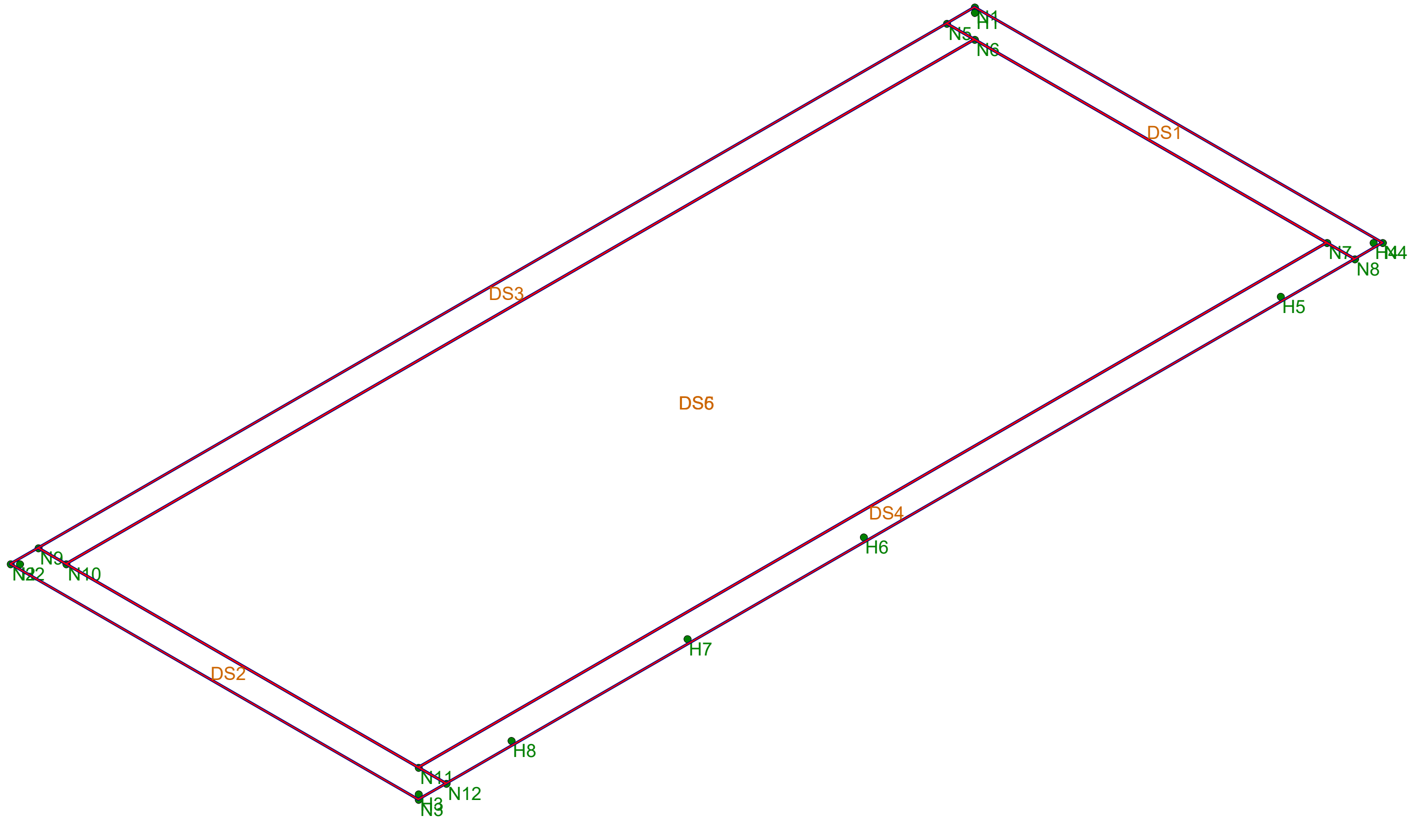
SK - 2

Apr 4, 2024 at 2:37 PM

Foundation.fnd

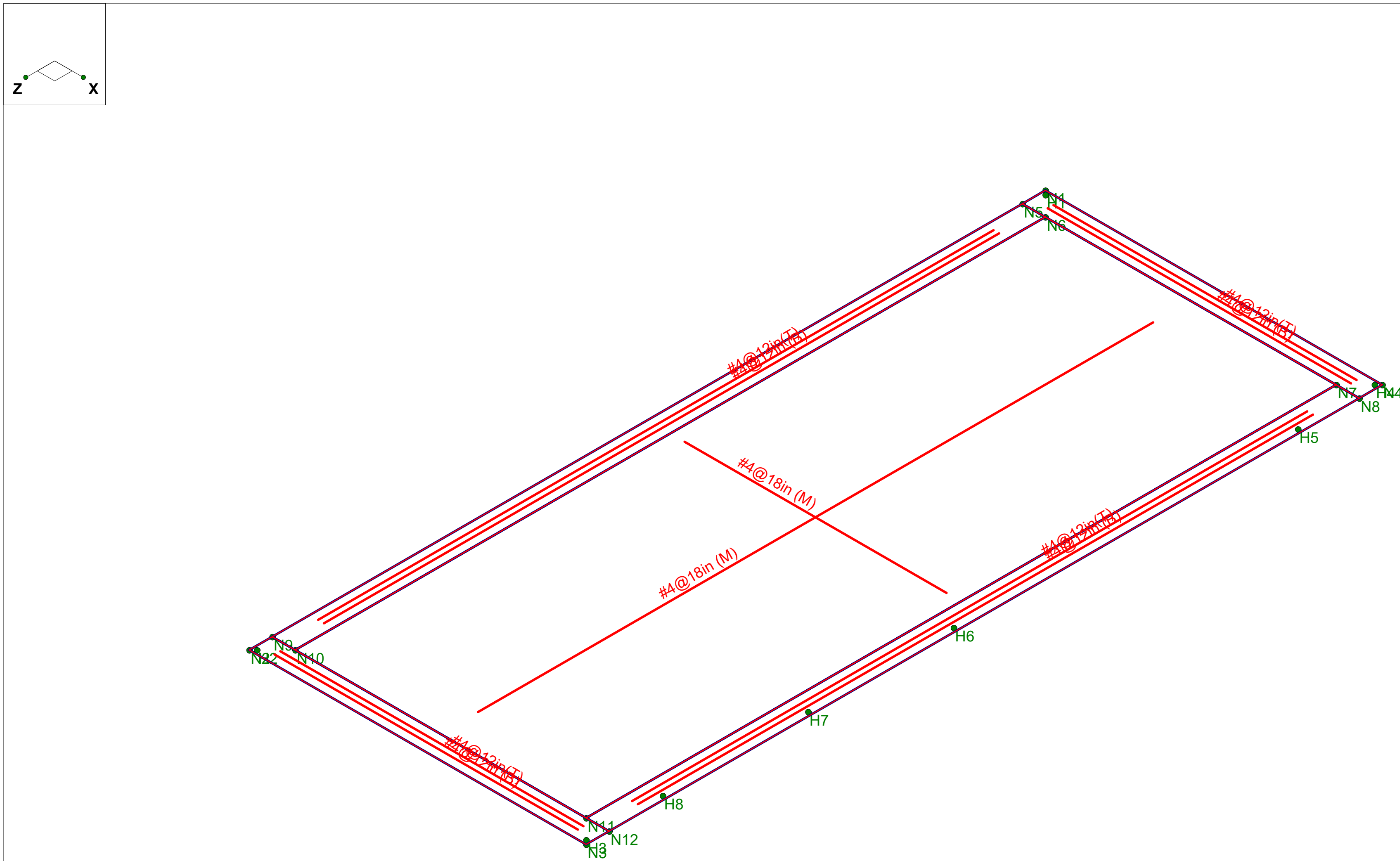
Envelope Soil Pressure





Loads: EPL - Earth Pressure Load
Enveloped Results

		SK - 3
		Apr 4, 2024 at 2:38 PM
	Design Strip Labels	Foundation.fnd

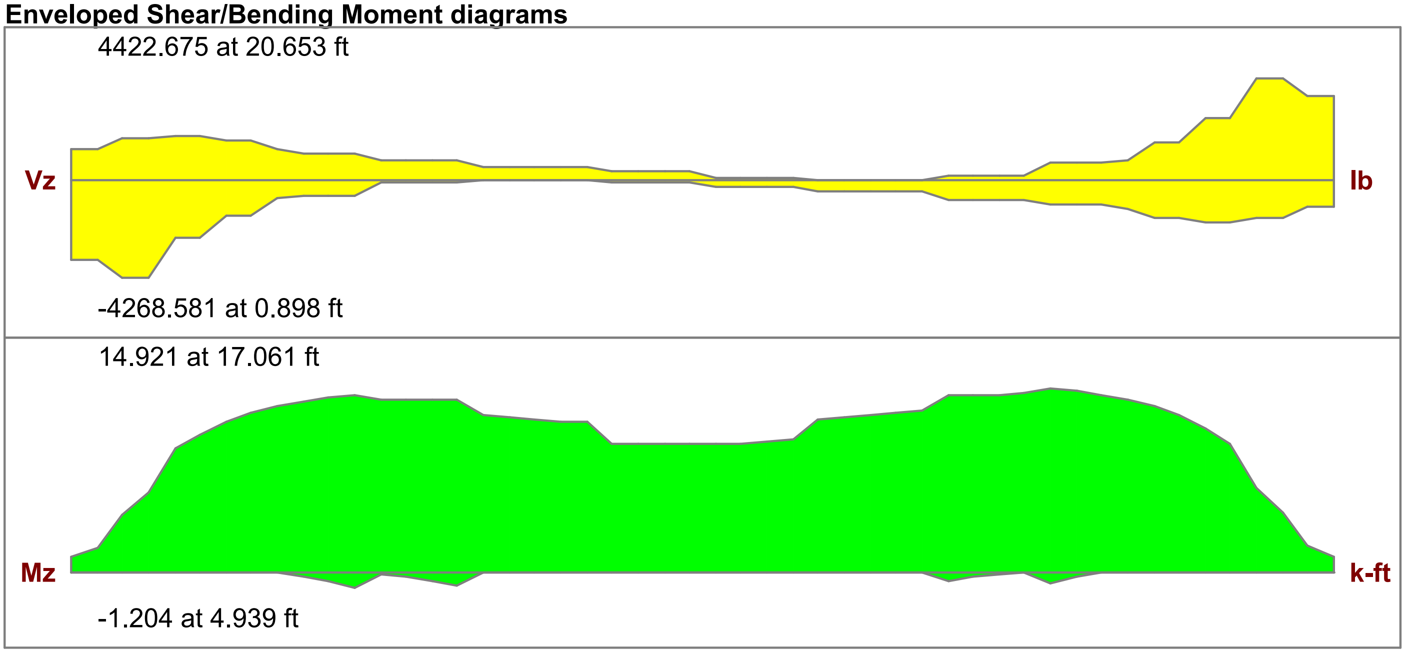


Loads: EPL - Earth Pressure Load
Enveloped Results

		SK - 4
		Apr 4, 2024 at 2:39 PM
	Design Rebar	Foundation.fnd

Strip:	DS1	Max Top bar Spac.:	18 in	Stress Block:	Rectangular
Material:	Conc4000NW	Min Top bar Spac.:	12 in	Rebar Orientation:	90
Strip Width:	18 in	Max Bot bar Spac.:	18 in	Rebar Spacing Inc:	2 in
Total Cuts:	50	Min Bot bar Spac.:	12 in	Design Rule:	Typical

Enveloped Shear/Bending Moment diagrams



ACI 318-19 Code Check

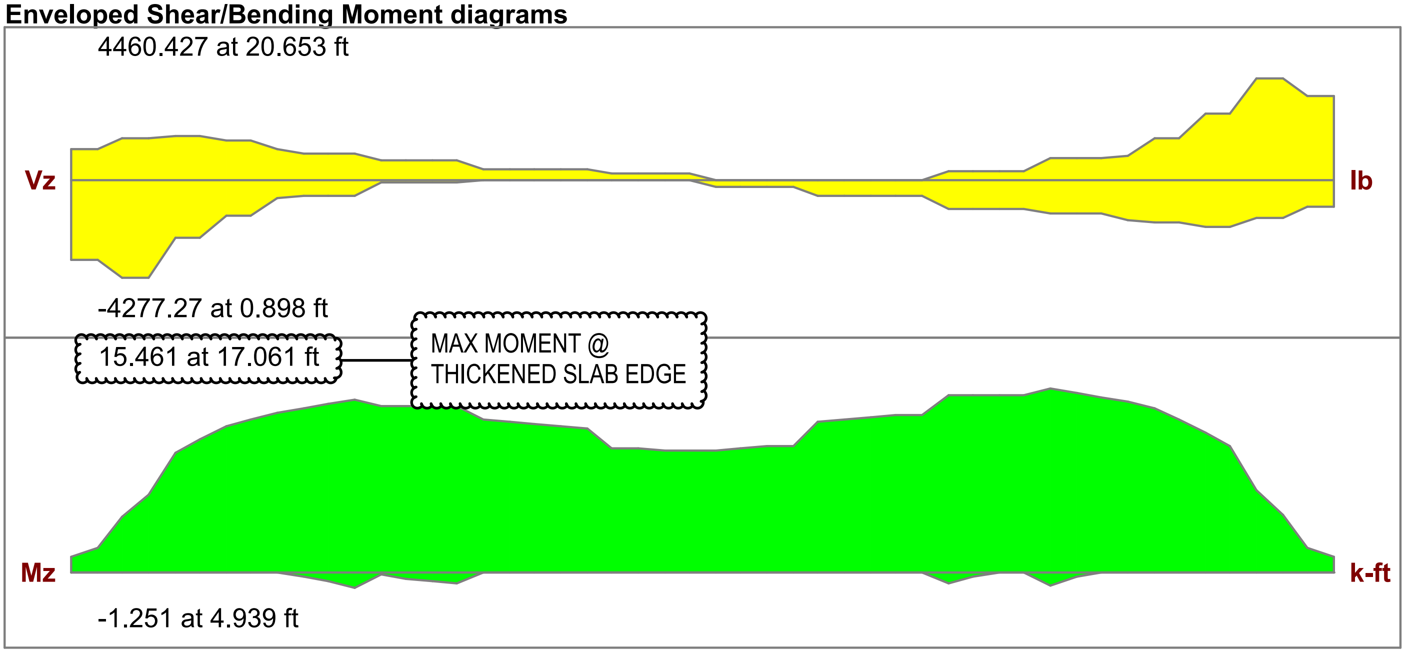
Top Bending Check	0.526	Bot Bending Check	0.047	1 Way Shear Check	0.385
Gov Mu Top	14.921 k-ft	Gov Mu Bot	-1.204 k-ft	Gov Vu	4422.675 lb
phi*Mn Top	28.376 k-ft	phi*Mn Bot	25.725 k-ft	phi*Vn	11479.326 lb
Governing Cut	DS1-X39	Governing Cut	DS1-X12	Governing Cut	DS1-X48
Governing LC	52	Governing LC	64	Governing LC	52
Tension Bar Fy	60 ksi	Concrete Weight	0.145 k/ft^3	Top Cover	1.5 in
Shear Bar Fy	60 ksi	λ	1	Bottom Cover	3 in
F'c	4 ksi	E_Concrete	3644 ksi	Side Cover	0 in
Flex. Rebar Set	ASTM A615	Rho Bot Prvd	0.00148	Rho Top Prvd	0.00134
		Prvd Bot Bar Spac.	#4@12in	Prvd Top Bar Spac.	#4@12in

Bending Steel Reqd/Prvd, Units: in^2)

Cut Label	Top As Reqd	Top As Prvd	Bot As Reqd	Bot As Prvd	Rho Reqd(T/S)	Rho Reqd(Flex)	Rho Prvd(Gross)
DS1-X39	0.205	0.393	0.012	0.393	0.00180	0.00180	0.00242
DS1-X12	0.198	0.393	0.018	0.393	0.00180	0.00180	0.00242

Strip:	DS2	Max Top bar Spac.:	18 in	Stress Block:	Rectangular
Material:	Conc4000NW	Min Top bar Spac.:	12 in	Rebar Orientation:	90
Strip Width:	18 in	Max Bot bar Spac.:	18 in	Rebar Spacing Inc:	2 in
Total Cuts:	50	Min Bot bar Spac.:	12 in	Design Rule:	Typical

Enveloped Shear/Bending Moment diagrams



ACI 318-19 Code Check

Top Bending Check	0.545	Bot Bending Check	0.049	1 Way Shear Check	0.389
Gov Mu Top	15.461 k-ft	Gov Mu Bot	-1.251 k-ft	Gov Vu	4460.427 lb
phi*Mn Top	28.376 k-ft	phi*Mn Bot	25.725 k-ft	phi*Vn	11479.325 lb
Governing Cut	DS2-X39	Governing Cut	DS2-X12	Governing Cut	DS2-X48
Governing LC	52	Governing LC	64	Governing LC	52

Tension Bar Fy	60 ksi	Concrete Weight	0.145 k/ft^3	Top Cover	1.5 in
Shear Bar Fy	60 ksi	λ	1	Bottom Cover	3 in
F'c	4 ksi	E_Concrete	3644 ksi	Side Cover	0 in
Flex. Rebar Set	ASTM A615	Rho Bot Prvd	0.00148	Rho Top Prvd	0.00134
		Prvd Bot Bar Spac.	#4@12in	Prvd Top Bar Spac.	#4@12in

Bending Steel Reqd/Prvd, Units: in^2)

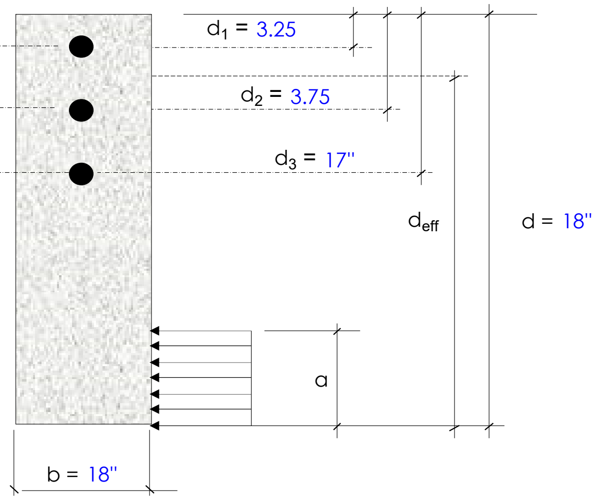
Cut Label	Top As Reqd	Top As Prvd	Bot As Reqd	Bot As Prvd	Rho Reqd(T/S)	Rho Reqd(Flex)	Rho Prvd(Gross)
DS2-X39	0.213	0.393	0.017	0.393	0.00180	0.00180	0.00242
DS2-X12	0.198	0.393	0.019	0.393	0.00180	0.00180	0.00242

Diagram/Input

A_{S1} = 0.40 sq. in f_{y1} = 60 ksi

A_{S2} = 0.00 sq. in f_{y2} = 60 ksi

A_{S3} = 0.00 sq. in f_{y3} = 60 ksi



Mu = 15.5 ft-k
f'_c = 6000 psi
ϕ = 0.90
B= 0.75

Results

$\phi M_n = \phi T \left(d_{eff} - \frac{a}{2} \right)$ = **26.3 ft-k** **> Mu = 15.5 ft-k** ***OK***

Maximum Reinforcing Check

ρ = 0.0015 < 0.75*p bal = 0.0283

Steel Strain = 0.1239 > 0.005 ACI-19

Minimum Reinforcing Check

ρ_min > 0.0039 or 4/3*ϕ Mn > Mu

Result Summary

Strength - OK

Maximum Reinforcing Ratio - OK

Minimum Reinforcing Ratio - OK

Calculations:

$T = A_{S1} f_{y1} + A_{S2} f_{y2} + A_{S3} f_{y3} =$ 24.00 k

$$d_{eff} = d - \left(\frac{A_{S1} f_{y1} d_1 + A_{S2} f_{y2} d_2 + A_{S3} f_{y3} d_3}{T} \right) = 18'' - \left(\frac{78.00 \text{ in-k}}{24.00 \text{ k}} \right) = 14.75''$$

$$a = \frac{T}{0.85 f'_c b} = \frac{24.00 \text{ k}}{91.80 \text{ k/in}} = 0.26''$$

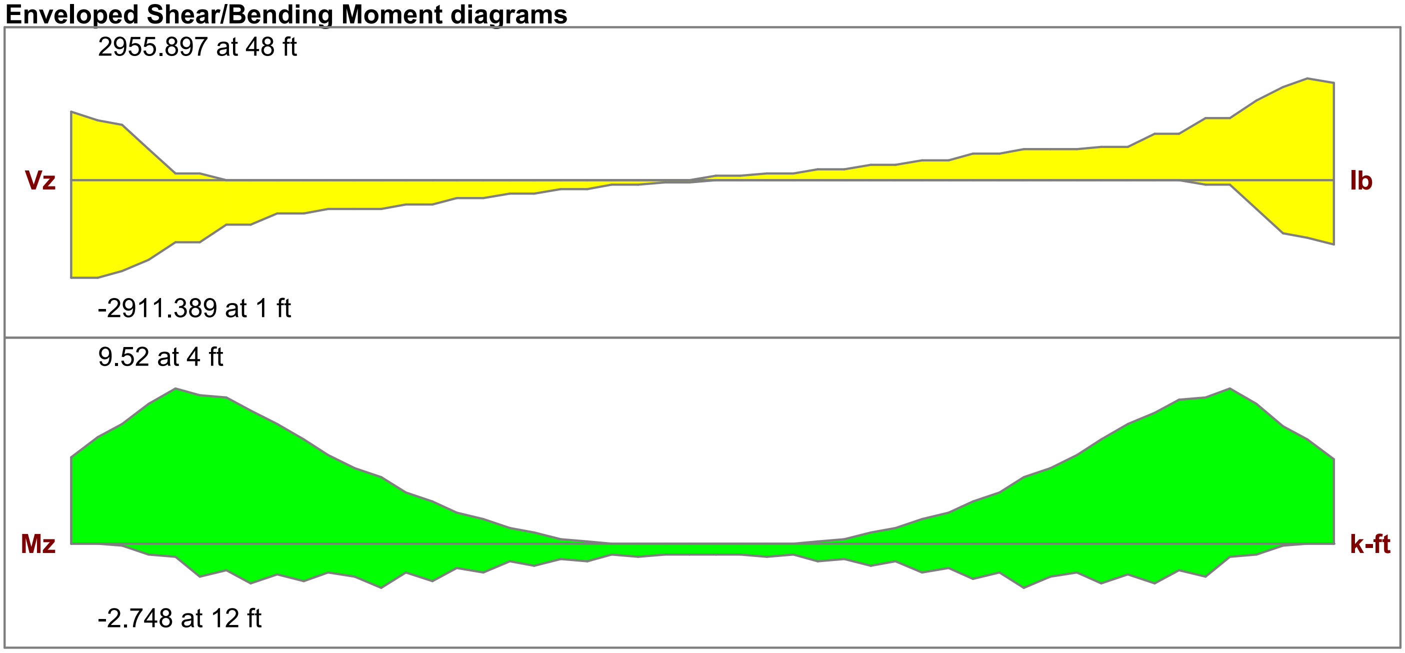
c= a/B1= 0.35"

Ku = Mu*12000/bd^2 = 47

Ku = ϕMn*12000/bd^2 = 81

Strip:	**DS3**	Max Top bar Spac.:	**18 in**	Stress Block:	**Rectangular**
Material:	**Conc4000NW**	Min Top bar Spac.:	**12 in**	Rebar Orientation:	**0**
Strip Width:	**18 in**	Max Bot bar Spac.:	**18 in**	Rebar Spacing Inc:	**2 in**
Total Cuts:	**50**	Min Bot bar Spac.:	**12 in**	Design Rule:	**Typical**

Enveloped Shear/Bending Moment diagrams



ACI 318-19 Code Check

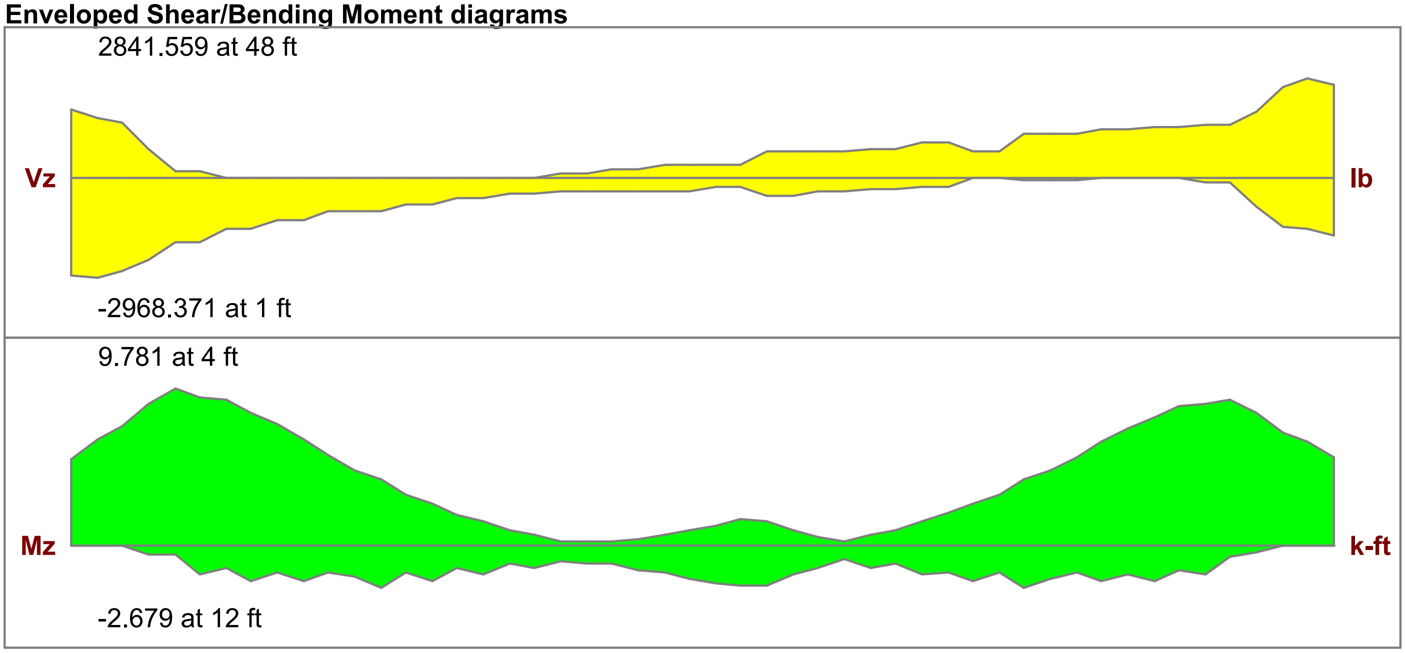
Top Bending Check	**0.335**	Bot Bending Check	**0.107**	1 Way Shear Check	**0.257**
Gov Mu Top	**9.52 k-ft**	Gov Mu Bot	**-2.748 k-ft**	Gov Vu	**2955.897 lb**
phi*Mn Top	**28.376 k-ft**	phi*Mn Bot	**25.725 k-ft**	phi*Vn	**11479.261 lb**
Governing Cut	**DS3-X5**	Governing Cut	**DS3-X13**	Governing Cut	**DS3-X49**
Governing LC	**54**	Governing LC	**52**	Governing LC	**52**
Tension Bar Fy	**60 ksi**	Concrete Weight	**0.145 k/ft^3**	Top Cover	**1.5 in**
Shear Bar Fy	**60 ksi**	λ	**1**	Bottom Cover	**3 in**
F'c	**4 ksi**	E_Concrete	**3644 ksi**	Side Cover	**0 in**
Flex. Rebar Set	**ASTM A615**	Rho Bot Prvd	**0.00148**	Rho Top Prvd	**0.00134**
		Prvd Bot Bar Spac.	**#4@12in**	Prvd Top Bar Spac.	**#4@12in**

Bending Steel Reqd/Prvd, Units: in^2)

Cut Label	Top As Reqd	Top As Prvd	Bot As Reqd	Bot As Prvd	Rho Reqd(T/S)	Rho Reqd(Flex)	Rho Prvd(Gross)
DS3-X5	**0.131**	**0.393**	**0.012**	**0.393**	**0.00180**	**0.00180**	**0.00242**
DS3-X13	**0.055**	**0.393**	**0.041**	**0.393**	**0.00180**	**0.00180**	**0.00242**

Strip:	**DS4**	Max Top bar Spac.:	**18 in**	Stress Block:	**Rectangular**
Material:	**Conc4000NW**	Min Top bar Spac.:	**12 in**	Rebar Orientation:	**0**
Strip Width:	**18 in**	Max Bot bar Spac.:	**18 in**	Rebar Spacing Inc:	**2 in**
Total Cuts:	**50**	Min Bot bar Spac.:	**12 in**	Design Rule:	**Typical**

Enveloped Shear/Bending Moment diagrams



ACI 318-19 Code Check

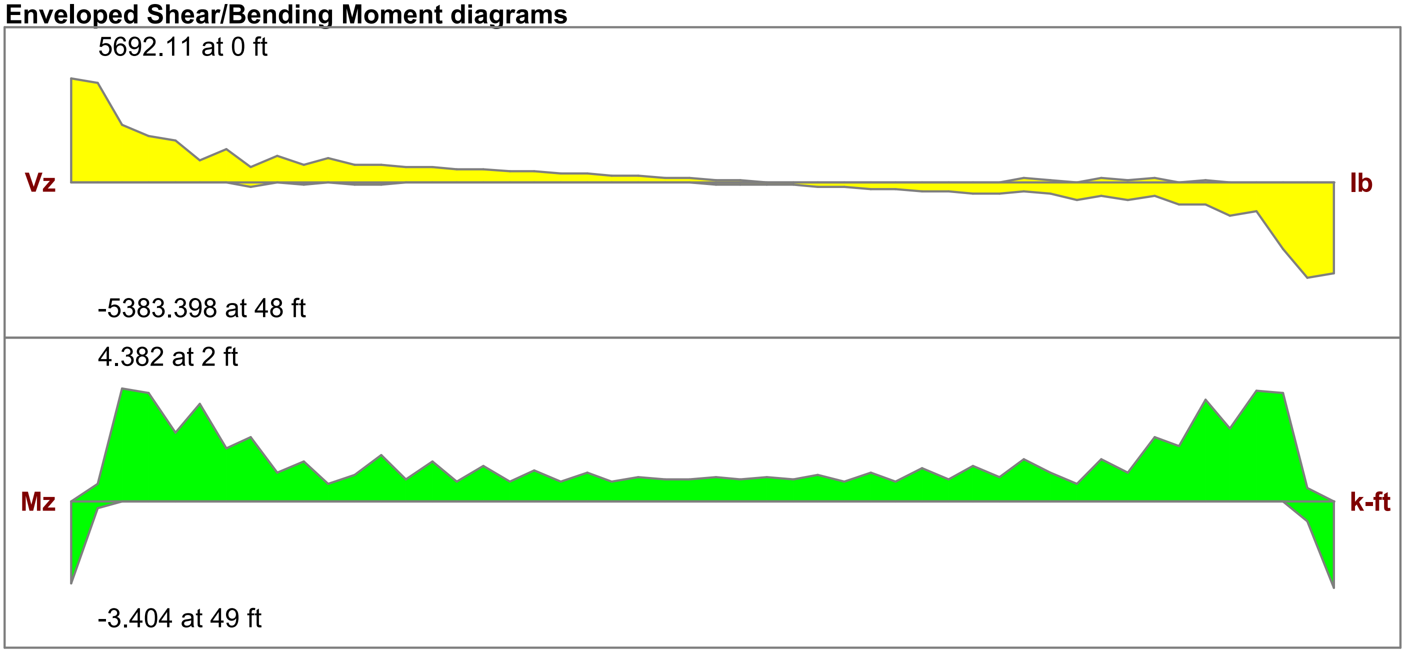
Top Bending Check	**0.345**	Bot Bending Check	**0.104**	1 Way Shear Check	**0.259**
Gov Mu Top	**9.781 k-ft**	Gov Mu Bot	**-2.679 k-ft**	Gov Vu	**2968.371 lb**
phi*Mn Top	**28.376 k-ft**	phi*Mn Bot	**25.725 k-ft**	phi*Vn	**11479.091 lb**
Governing Cut	**DS4-X5**	Governing Cut	**DS4-X13**	Governing Cut	**DS4-X2**
Governing LC	**52**	Governing LC	**54**	Governing LC	**54**
Tension Bar Fy	**60 ksi**	Concrete Weight	**0.145 k/ft^3**	Top Cover	**1.5 in**
Shear Bar Fy	**60 ksi**	λ	**1**	Bottom Cover	**3 in**
F'c	**4 ksi**	E_Concrete	**3644 ksi**	Side Cover	**0 in**
Flex. Rebar Set	**ASTM A615**	Rho Bot Prvd	**0.00148**	Rho Top Prvd	**0.00134**
		Prvd Bot Bar Spac.	**#4@12in**	Prvd Top Bar Spac.	**#4@12in**

Bending Steel Reqd/Prvd, Units: in^2)

Cut Label	Top As Reqd	Top As Prvd	Bot As Reqd	Bot As Prvd	Rho Reqd(T/S)	Rho Reqd(Flex)	Rho Prvd(Gross)
DS4-X5	**0.134**	**0.393**	**0.009**	**0.393**	**0.00180**	**0.00180**	**0.00242**
DS4-X13	**0.055**	**0.393**	**0.04**	**0.393**	**0.00180**	**0.00180**	**0.00242**

Strip:	DS5 ← SOG	Max Top bar Spac.:	18 in	Stress Block:	Rectangular
Material:	Conc4000NW	Min Top bar Spac.:	12 in	Rebar Orientation:	0
Strip Width:	228 in	Max Bot bar Spac.:	NA	Rebar Spacing Inc:	2 in
Total Cuts:	50	Min Bot bar Spac.:	NA	Design Rule:	SOG

Enveloped Shear/Bending Moment diagrams



ACI 318-19 Code Check

Top Bending Check	0.159	Bot Bending Check	0.123	1 Way Shear Check	0.160
Gov Mu Top	4.382 k-ft	Gov Mu Bot	-3.404 k-ft	Gov Vu	5692.11 lb
phi*Mn Top	27.581 k-ft	phi*Mn Bot	27.581 k-ft	phi*Vn	35652.424 lb
Governing Cut	DS5-X3	Governing Cut	DS5-X50	Governing Cut	DS5-X1
Governing LC	33	Governing LC	55	Governing LC	33

Tension Bar Fy	60 ksi	Concrete Weight	0.145 k/ft^3
Shear Bar Fy	60 ksi	λ	1
F'c	4 ksi	E_Concrete	3644 ksi
Flex. Rebar Set	ASTM A615	Rho Mid Prvd	0.00448
		Prvd Mid Bar Spac.	#4@18in

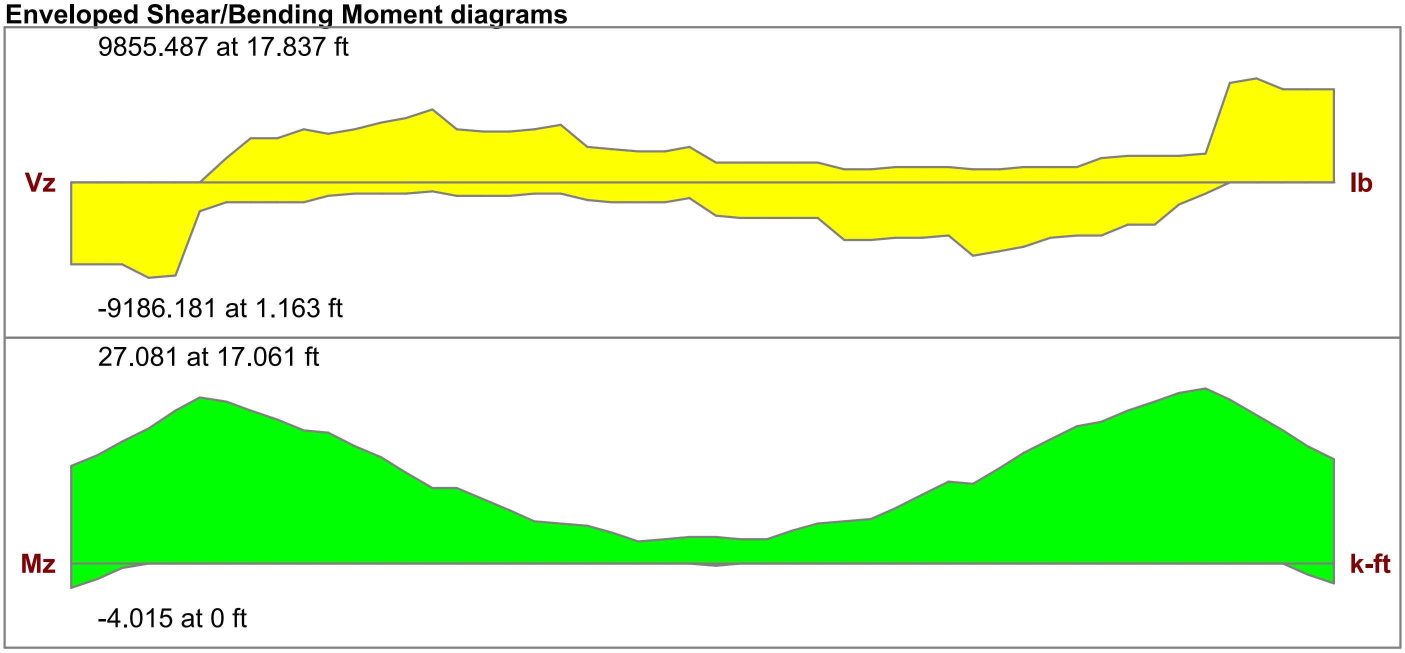
Bending Steel Reqd/Prvd, Units: in^2)

Cut Label	Top As Reqd	Top As Prvd	Mid As Reqd	Mid As Prvd	Rho Reqd(T/S)	Rho Reqd(Flex)	Rho Prvd(Gross)
DS5-X3	0.304	2.553	NA	NA	0.00180	0.00180	0.00224
DS5-X50	0.304	2.553	NA	NA	0.00180	0.00180	0.00224

Company	:		Apr 4, 2024
Designer	:		2:53 PM
Job Number	:		Checked By:______
Model Name	:		

Strip:	**DS6** ← SOG	Max Top bar Spac.:	**18 in**	Stress Block:	**Rectangular**
Material:	**Conc4000NW**	Min Top bar Spac.:	**12 in**	Rebar Orientation:	**90**
Strip Width:	**588 in**	Max Bot bar Spac.:	**NA**	Rebar Spacing Inc:	**2 in**
Total Cuts:	**50**	Min Bot bar Spac.:	**NA**	Design Rule:	**SOG**

Enveloped Shear/Bending Moment diagrams



ACI 318-19 Code Check

Top Bending Check	**0.387**	Bot Bending Check	**0.057**	1 Way Shear Check	**0.108**
Gov Mu Top	**27.081 k-ft**	Gov Mu Bot	**-4.015 k-ft**	Gov Vu	**9855.487 lb**
phi*Mn Top	**70.06 k-ft**	phi*Mn Bot	**70.06 k-ft**	phi*Vn	**91462.079 lb**
Governing Cut	**DS6-X45**	Governing Cut	**DS6-X1**	Governing Cut	**DS6-X47**
Governing LC	**52**	Governing LC	**64**	Governing LC	**54**

Tension Bar Fy	**60 ksi**	Concrete Weight	**0.145 k/ft^3**
Shear Bar Fy	**60 ksi**	λ	**1**
F'c	**4 ksi**	E_Concrete	**3644 ksi**
Flex. Rebar Set	**ASTM A615**	Rho Mid Prvd	**0.00441**
		Prvd Mid Bar Spac.	**#4@18in**

Bending Steel Reqd/Prvd, Units: in^2)

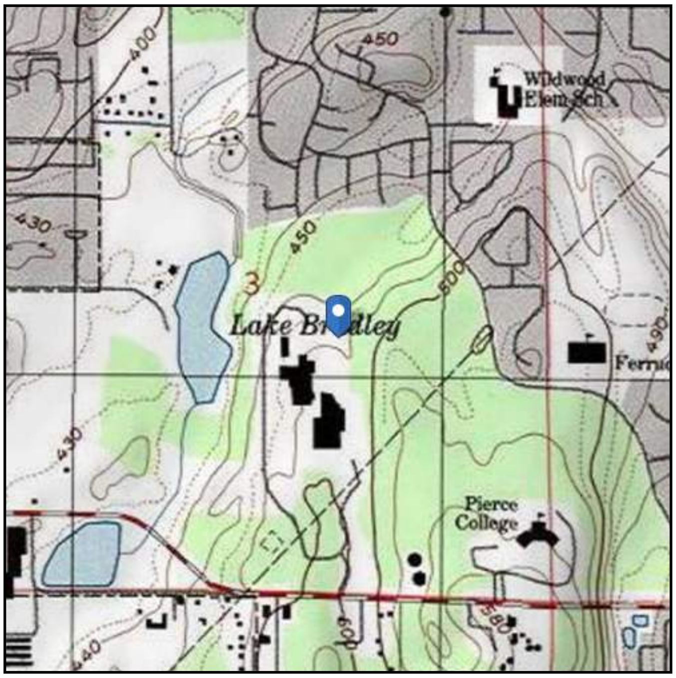
Cut Label	Top As Reqd	Top As Prvd	Mid As Reqd	Mid As Prvd	Rho Reqd(T/S)	Rho Reqd(Flex)	Rho Prvd(Gross)
DS6-X45	**1.357**	**6.48**	**NA**	**NA**	**0.00180**	**0.00180**	**0.00220**
DS6-X1	**1.357**	**6.48**	**NA**	**NA**	**0.00180**	**0.00180**	**0.00220**

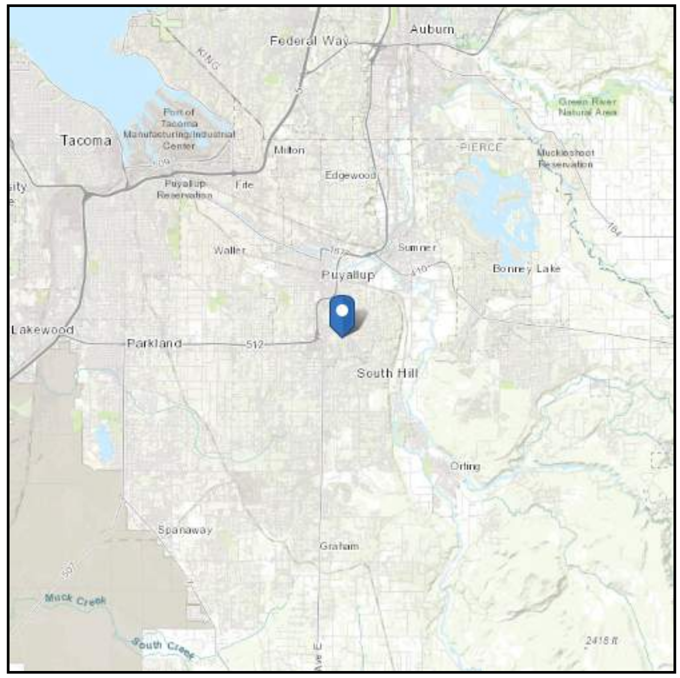
Lateral Design

ASCE Hazards Report

Address:
1023 39th Ave SE
Puyallup, Washington
98374

Standard:	ASCE/SEI 7-16	**Latitude:**	47.160853
Risk Category:	II	**Longitude:**	-122.279318
Soil Class:	D - Default (see Section 11.4.3)	**Elevation:**	482.88472036372787 ft (NAVD 88)





Wind

Results:

Wind Speed	98 Vmph
10-year MRI	67 Vmph
25-year MRI	73 Vmph
50-year MRI	78 Vmph
100-year MRI	83 Vmph

Data Source:	ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed:	Mon Feb 05 2024

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

Seismic

Site Soil Class: D - Default (see Section 11.4.3)

Results:

S_S :	1.257	S_{D1} :	N/A
S_1 :	0.434	T_L :	6
F_a :	1.2	PGA :	0.5
F_v :	N/A	PGA_M :	0.6
S_{MS} :	1.509	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1
S_{DS} :	1.006	C_v :	1.351

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Mon Feb 05 2024

Date Source: USGS Seismic Design Maps

MecaWind v2462

Calculations Prepared by:
Date: Feb 08, 2024
File Location: G:\2024\24202 Centeris Shed\Calcs\Centris Wind.wnd

General:

Wind Load Standard	=	ASCE 7-16	Basic Wind Speed	=	98.0 mph
Exposure Classification	=	B	Risk Category	=	II
Structure Type	=	Building	Design Basis for Wind Pressures	=	LRFD
MWFRS Analysis Method	=	Ch 27 Pt 1	C&C Analysis Method	=	Ch 30 Pt 1
Dynamic Type of Structure	=	Rigid	Show Advanced Options	=	-1
Reset Advanced Options to Default Values	=	Defaults	Simple Diaphragm Building	=	0
Show Base Reactions in Output	=	Summary	Altitude above Sea Level	=	482.900 ft
Base Elevation Of Structure	=	0.000 ft	MWFRS Pressure Elevations	=	Mean Ht
Topographic Effects	=	None	Override Directionality Factor K_d	=	0
Override the Gust Factor G	=	0	Override Minimum Pressure	=	0

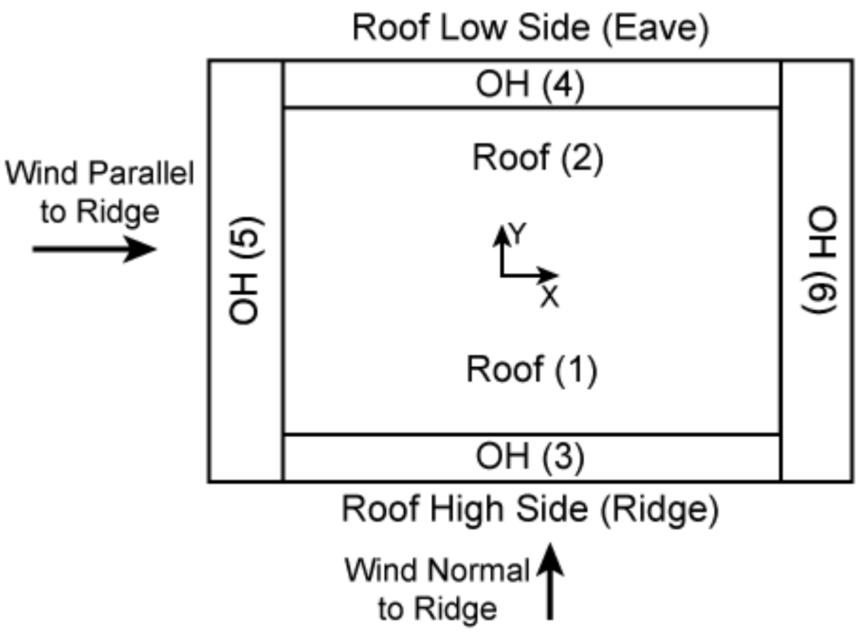
Building:

Roof	=	Roof Type	=	Monoslope	Encl	=	Enclosure Classification	=	Enclosed
Help	=	Help on Building Roof Type	=	Help	Pitch	=	Pitch of Roof	=	2.0 :12
θ	=	Slope of Roof	=	9.46 Deg	R_{Ht}	=	Ridge Height	=	23.161 ft
E_{Ht}	=	Eave Height	=	14.830 ft	W	=	Building Width	=	22.000 ft
L	=	Building Length	=	52.000 ft	OH	=	Type of Overhang	=	None
Par	=	Parapet	=	None	HT_{over}	=	Override Mean Roof Height	=	0
Ht_{man}	=	Mean Roof Height	=	14.830 ft	RA_{over}	=	Override Roof Area	=	0
GC_{pi_o}	=	Override GC_{pi}value	=	0		=			

Exposure Constants [Tbl 26.11-1]:

α = 3-s Gust-speed exponent	=	7.000	Z_g= Nominal Ht of Boundary Layer	=	1200.000 ft
â = Reciprocol of α	=	0.143	b = 3 sec gust speed factor	=	0.840
α_m= Mean hourly Wind-Speed Exponent	=	0.250	b_m= Mean hourly Windspeed Exponent	=	0.450
c = Turbulence Intensity Factor	=	0.300	ε = Integral Length Scale Exponent	=	0.3333

Main Wind Force Resisting System (MWFRS) Wind Calculations per Ch 27 Pt1



h	=	Mean structure height	=	14.830 ft
h_{grade}	=	Elevation from Grade to Top of Structure	=	14.830 ft
K_h	=	$2.01\bullet(15/Z_g)^{2/\alpha}$[Tbl 26.10-1]	=	0.575
K_{zt}	=	No Topographic feature specified	=	1.000
K_d	=	Wind Directionality Factor per Tbl 26.6-1	=	0.85
$+GC_{pi}$	=	Enclosed Positive Internal Pressure Tbl 26.13-1	=	+0.18
$-GC_{pi}$	=	Enclosed Negative Internal Pressure Tbl 26.13-1	=	-0.18
LF	=	Load Factor based upon STRENGTH Design	=	1.00
K_e	=	Ground Elev Factor [Tbl 26.9-1]	=	0.983
q_h	=	$0.00256\bullet K_h\bullet K_{zt}\bullet K_d\bullet K_e\bullet V^2 * LF$ [Eq 26.10-1]	=	11.80 psf
RA	=	Roof Area	=	1223.28 ft^2

q_h = $0.00256 \bullet K_h \bullet K_{zt} \bullet K_d \bullet K_e \bullet V^2 * LF$ [Eq 26.10-1] = 11.80 psf
q_{in} = Negative Internal Pressure: $q_h \bullet LF$ = 11.80 psf
q_{ip} = Positive Internal Pressure: $q_h \bullet LF$ = 11.80 psf

MWFRS Wind Loads [Normal to Ridge]

h = Mean Roof Height Of Building = 14.830 ft
RHt = Ridge Height Of Roof = 23.161 ft
B = Horizontal Dimension Of Building Normal To Wind Direction = 52.000 ft
L = Horizontal Dimension Of building Parallel To Wind Direction = 22.000 ft
L/B = Ratio Of L/B used For Cp determination = 0.423
h/L = Ratio Of h/L used For Cp determination = 0.674
Slope = Slope Of Roof = 9.46 Deg

Gust Factor Calculation for Wind: [Normal to Ridge]

Gust Factor Category I Rigid Structures - Simplified Method
G_1 = Simplified: For Rigid Structures can use 0.85 = 0.85
Gust Factor Category II Rigid Structures - Complete Analysis
Z_m = Equiv Struc Height: Max(0.6•h, Z_{min}) = 30.000 ft
I_{zm} = Turbulence Intensity: $c \bullet (33/Z_m)^{1/6}$ [Eq 26.11-1] = 0.305
L_{zm} = Turbulence Integral Length Scale: $\ell \bullet (Z_m/33)^{\varepsilon}$ [Eq 26.11-9] = 309.993 ft
B = Building Width Width Normal to Wind Direction = 52.000 ft
Q = $[1/(1+0.63 \bullet [(B+h)/L_{zm}]^{0.63})]^{0.5}$ [Eq 26.11-8] = 0.898
G_2 = Detailed: $0.925 \bullet [(1+1.7 \bullet g_q \bullet I_{zm} \bullet Q)/(1+1.7 \bullet g_v \bullet I_{zm})]$ [Eq 26.11-6] = 0.865
Gust Factor Used in Analysis
G = Gust Factor: Min(G_1, G_2) = 0.850

Cp_{WW} = Windward Wall Coefficient (All L/B Values) = 0.800
Cp_{LW} = Leeward Wall Coefficient using L/B = -0.500
Cp_{SW} = Side Wall Coefficient (All L/B values) = -0.700

Wind Pressures [Normal to Ridge]
All wind pressures include a Load Factor (LF) of 1.0

Elev ft	GC_{pi} Windward	GC_{pi} Leeward	q_i psf	K_z	K_{zt}	q_z psf	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
23.161	+0.18	+0.18	11.80	0.651	1.000	13.36	6.96	-7.14	-9.15	14.10	16.00
14.830	+0.18	+0.18	11.80	0.575	1.000	11.80	5.90	-7.14	-9.15	13.04	16.00
23.161	-0.18	-0.18	11.80	0.651	1.000	13.36	11.21	-2.89	-4.90	14.10	16.00
14.830	-0.18	-0.18	11.80	0.575	1.000	11.80	10.15	-2.89	-4.90	13.04	16.00

K_z = $2.01 \bullet (15/Z_g)^{2/\alpha}$ [Tbl 26.10-1]
GC_{pi} = Enclosed Internal Pressure Tbl 26.13-1
q_{ip} = Positive Internal Pressure: $q_h \bullet LF$
Side = $q_h \bullet G \bullet Cp_{SW} - q_{ip} \bullet (+GC_{pi})$ [Eq 27.3-1]
Windward = $q_z \bullet G \bullet Cp_{WW} - q_{ip} \bullet (+GC_{pi})$ [Eq 27.3-1]
K_{zt} = No Topographic feature specified
q_z = $0.00256 \bullet K_z \bullet K_{zt} \bullet K_d \bullet K_e \bullet V^2 * LF$ [Eq 26.10-1]
q_{in} = Negative Internal Pressure: $q_h \bullet LF$
Leeward = $q_h \bullet G \bullet Cp_{LW} - q_{ip} \bullet (+GC_{pi})$ [Eq 27.3-1]
Total = Windward - Leeward

- *Minimum Pressure: § 27.1.5 no less than 16.00 psf (Incl LF) applied to Walls*
- *Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface*

Roof Wind Pressures for Positive & Negative Internal Pressure (±GC_{pi}) [Normal to Ridge]
All wind pressures include a Load Factor (LF) of 1.0

Reference	Description	Location	Start Dist ft	End Dist ft	C_p Min	C_p Max	GC_{pi}	Pressure Min psf	Pressure Max psf
Roof	Roof (0 to h/2)	All	0.000	7.415	-0.949	-0.180	+0.18/-0.18	-11.64	8.00
Roof	Roof (h/2 to h)	All	7.415	14.830	-0.830	-0.180	+0.18/-0.18	-10.45	8.00
Roof	Roof (h to 2*h)	All	14.830	22.000	-0.570	-0.180	+0.18/-0.18	-8.00	8.00

Roof Pressures based upon Ch 27 Pt1:
Start = Start Dist from Windward Edge
C_{p_min} = Smallest Coefficient Magnitude
$Press_{Min}$ = $q_h \bullet G \bullet C_{p\_min} - q_{ip} \bullet (+GC_{pi})$ Eq 27.3-1
End = End Dist from Windward Edge
C_{p_max} = Largest Coefficient Magnitude
$Press_{Max}$ = $q_h \bullet G \bullet C_{p\_max} - q_{in} \bullet (-GC_{pi})$ Eq 27.3-1

- *0.800 Reduction Factor applied for h/L>=1 & Slope10 Deg*
- *The smaller uplift pressures due to Cp_Min can become critical when wind is combined with roof live load or snow load; load combinations are given in ASCE 7*
- *Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface*

MWFRS Wind Loads [Normal to Eave]

h = Mean Roof Height Of Building = 14.830 ft
RHt = Ridge Height Of Roof = 23.161 ft
B = Horizontal Dimension Of Building Normal To Wind Direction = 52.000 ft
L = Horizontal Dimension Of building Parallel To Wind Direction = 22.000 ft
L/B = Ratio Of L/B used For Cp determination = 0.423
h/L = Ratio Of h/L used For Cp determination = 0.674
Slope = Slope Of Roof = 9.46 Deg

Gust Factor Calculation for Wind: [Normal to Eave]

Gust Factor Category I Rigid Structures - Simplified Method

G_1 = Simplified: For Rigid Structures can use 0.85 = 0.85

Gust Factor Category II Rigid Structures - Complete Analysis

Z_m = Equiv Struc Height: Max(0.6•h, Z_{min}) = 30.000 ft
I_{zm} = Turbulence Intensity: $c•(33/Z_m)^{1/6}$[Eq 26.11-1] = 0.305
L_{zm} = Turbulence Integral Length Scale: $\ell•(Z_m/33)^{\varepsilon}$[Eq 26.11-9] = 309.993 ft
B = Building Width Width Normal to Wind Direction = 52.000 ft
Q = $[1/(1+0.63•[(B+h)/L_{zm}]^{0.63})]^{0.5}$[Eq 26.11-8] = 0.898
G_2 = Detailed: $0.925•[(1+1.7•g_q•I_{zm}•Q)/(1+1.7•g_v•I_{zm})]$ [Eq 26.11-6] = 0.865

Gust Factor Used in Analysis

G = Gust Factor: Min(G_1, G_2) = 0.850

Cp_{WW} = Windward Wall Coefficient (All L/B Values) = 0.800
Cp_{LW} = Leeward Wall Coefficient using L/B = -0.500
Cp_{SW} = Side Wall Coefficient (All L/B values) = -0.700

Wind Pressures [Normal to Eave]
All wind pressures include a Load Factor (LF) of 1.0

Elev ft	GC_{pi} Windward	GC_{pi} Leeward	q_i psf	K_z	K_{zt}	q_z psf	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
14.830	+0.18	+0.18	11.80	0.575	1.000	11.80	5.90	-7.14	-9.15	13.04	16.00
14.830	-0.18	-0.18	11.80	0.575	1.000	11.80	10.15	-2.89	-4.90	13.04	16.00

K_z = $2.01•(15/Z_g)^{2/\alpha}$[Tbl 26.10-1]
GC_{pi} = Enclosed Internal Pressure Tbl 26.13-1
q_{ip} = Positive Internal Pressure: q_h•LF
Side = $q_h•G•Cp_{SW}-q_{ip}•(+GC_{pi})$ [Eq 27.3-1]
Windward = $q_z•G•Cp_{WW}-q_{ip}•(+GC_{pi})$ [Eq 27.3-1]
K_{zt} = No Topographic feature specified
*q_z = $0.00256•K_z•K_{zt}•K_d•K_e•V^2$*LF [Eq 26.10-1]*
q_{in} = Negative Internal Pressure: q_h•LF
Leeward = $q_h•G•Cp_{LW}-q_{ip}•(+GC_{pi})$ [Eq 27.3-1]
Total = Windward - Leeward

- *Minimum Pressure: § 27.1.5 no less than 16.00 psf (Incl LF) applied to Walls*
- *Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface*

Roof Wind Pressures for Positive & Negative Internal Pressure (±GC_{pi}) [Normal to Eave]
All wind pressures include a Load Factor (LF) of 1.0

Reference	Description	Location	Start Dist ft	End Dist ft	C_p Min	C_p Max	GC_{pi}	Pressure Min psf	Pressure Max psf
Roof	Roof (0 to h/2)	All	0.000	7.415	-0.949	-0.180	+0.18/-0.18	-11.64	8.00
Roof	Roof (h/2 to h)	All	7.415	14.830	-0.830	-0.180	+0.18/-0.18	-10.45	8.00
Roof	Roof (h to 2*h)	All	14.830	22.000	-0.570	-0.180	+0.18/-0.18	-8.00	8.00

Roof Pressures based upon Ch 27 Pt1:
Start = Start Dist from Windward Edge
C_{p_min} = Smallest Coefficient Magnitude
$Press_{Min}$ = $q_h•G•C_{p\_min}-q_{ip}•(+GC_{pi})$ Eq 27.3-1
End = End Dist from Windward Edge
C_{p_max} = Largest Coefficient Magnitude
$Press_{Max}$ = $q_h•G•C_{p\_max}-q_{in}•(-GC_{pi})$ Eq 27.3-1

- *0.800 Reduction Factor applied for h/L>=1 & (0 to h/2)*
- *The smaller uplift pressures due to Cp_Min can become critical when wind is combined with roof live load or snow load; load combinations are given in ASCE 7*
- *Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface*

MWFRS Wind Loads [Parallel to Ridge]

h = Mean Roof Height Of Building = 14.830 ft
RHt = Ridge Height Of Roof = 23.161 ft
B = Horizontal Dimension Of Building Normal To Wind Direction = 22.000 ft
L = Horizontal Dimension Of building Parallel To Wind Direction = 52.000 ft
L/B = Ratio Of L/B used For Cp determination = 2.364
h/L = Ratio Of h/L used For Cp determination = 0.285
Slope = Slope Of Roof = 9.46 Deg

Gust Factor Calculation for Wind: [Parallel to Ridge]

Gust Factor Category I Rigid Structures - Simplified Method

G_1 = Simplified: For Rigid Structures can use 0.85 = 0.85

Gust Factor Category II Rigid Structures - Complete Analysis

Z_m = Equiv Struc Height: Max(0.6•h, Z_{min}) = 30.000 ft
I_{zm} = Turbulence Intensity: $c•(33/Z_m)^{1/6}$[Eq 26.11-1] = 0.305
L_{zm} = Turbulence Integral Length Scale: $\ell•(Z_m/33)^{\varepsilon}$[Eq 26.11-9] = 309.993 ft
B = Building Width Width Normal to Wind Direction = 22.000 ft
Q = $[1/(1+0.63•[(B+h)/L_{zm}]^{0.63})]^{0.5}$[Eq 26.11-8] = 0.927
G_2 = Detailed: $0.925•[(1+1.7•g_q•I_{zm}•Q)/(1+1.7•g_v•I_{zm})]$ [Eq 26.11-6] = 0.882

Gust Factor Used in Analysis

G = Gust Factor: Min(G_1, G_2) = 0.850

Cp_{WW} = Windward Wall Coefficient (All L/B Values) = 0.800
Cp_{LW} = Leeward Wall Coefficient using L/B = -0.282
Cp_{SW} = Side Wall Coefficient (All L/B values) = -0.700

Wind Pressures [Parallel to Ridge]
All wind pressures include a Load Factor (LF) of 1.0

Elev ft	GC_{pi} Windward	GC_{pi} Leeward	q_i psf	K_z	K_{zt}	q_z psf	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
23.161	+0.18	+0.18	11.80	0.651	1.000	13.36	6.96	-4.95	-9.15	11.91	16.00
14.830	+0.18	+0.18	11.80	0.575	1.000	11.80	5.90	-4.95	-9.15	10.85	16.00
23.161	-0.18	-0.18	11.80	0.651	1.000	13.36	11.21	-0.70	-4.90	11.91	16.00
14.830	-0.18	-0.18	11.80	0.575	1.000	11.80	10.15	-0.70	-4.90	10.85	16.00

K_z = $2.01\bullet(15/Z_g)^{2/\alpha}$ [Tbl 26.10-1]
GC_{pi} = Enclosed Internal Pressure Tbl 26.13-1
q_{ip} = Positive Internal Pressure: $q_h\bullet LF$
Side = $q_h\bullet G\bullet Cp_{SW}-q_{ip}\bullet(+GC_{pi})$ [Eq 27.3-1]
Windward = $q_z\bullet G\bullet Cp_{WW}-q_{ip}\bullet(+GC_{pi})$ [Eq 27.3-1]
K_{zt} = No Topographic feature specified
q_z = $0.00256\bullet K_z\bullet K_{zt}\bullet K_d\bullet K_e\bullet V^2*LF$ [Eq 26.10-1]
q_{in} = Negative Internal Pressure: $q_h\bullet LF$
Leeward = $q_h\bullet G\bullet Cp_{LW}-q_{ip}\bullet(+GC_{pi})$ [Eq 27.3-1]
Total = Windward - Leeward

- *Minimum Pressure: § 27.1.5 no less than 16.00 psf (Incl LF) applied to Walls*
- *Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface*

Roof Wind Pressures for Positive & Negative Internal Pressure (±GC_{pi}) [Parallel to Ridge]
All wind pressures include a Load Factor (LF) of 1.0

Reference	Description	Location	Start Dist ft	End Dist ft	C_p Min	C_p Max	GC_{pi}	Pressure Min psf	Pressure Max psf
Roof	Roof (0 to h)	All	0.000	14.830	-0.900	-0.180	+0.18/-0.18	-11.15	8.00
Roof	Roof (h to 2*h)	All	14.830	29.660	-0.500	-0.180	+0.18/-0.18	-8.00	8.00
Roof	Roof (>= 2*h)	All	29.660	52.000	-0.300	-0.180	+0.18/-0.18	-8.00	8.00

Roof Pressures based upon Ch 27 Pt1:
Start = Start Dist from Windward Edge
C_{p_min} = Smallest Coefficient Magnitude
$Press_{Min}$ = $q_h\bullet G\bullet C_{p\_min}-q_{ip}\bullet(+GC_{pi})$ Eq 27.3-1
End = End Dist from Windward Edge
C_{p_max} = Largest Coefficient Magnitude
$Press_{Max}$ = $q_h\bullet G\bullet C_{p\_max}-q_{in}\bullet(-GC_{pi})$ Eq 27.3-1

- *No reduction factor was applicable*
- *The smaller uplift pressures due to Cp_Min can become critical when wind is combined with roof live load or snow load; load combinations are given in ASCE 7*
- *Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface*

Reaction Summary Wind (MWFRS)

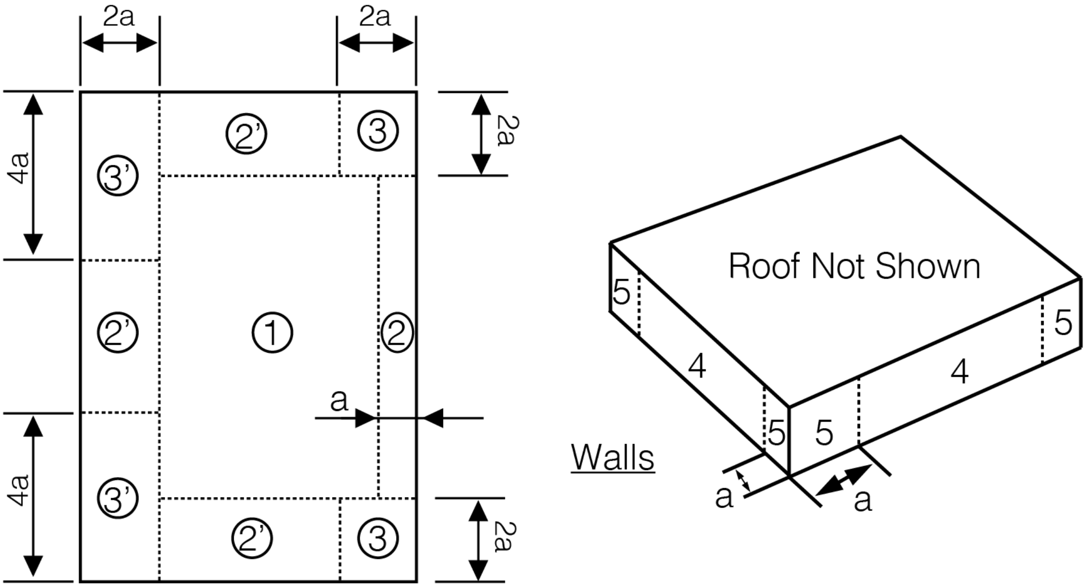
Description	F_x Kip	F_y Kip	F_z Kip	M_x k•ft	M_y k•ft	M_z k•ft
Normal to Ridge: Walls+Roof|+GCPi	0.00	-17.41	-11.44	226.26	0.00	0.00
Normal to Ridge: Walls Only|+GCPi	0.00	-13.07	0.00	131.86	0.00	0.00
Normal to Ridge: Walls+Roof|-GCPi	0.00	-14.78	0.36	164.21	0.00	0.00
Normal to Ridge: Walls Only|-GCPi	0.00	-14.91	0.00	166.83	0.00	0.00
Normal to Ridge: Walls+Roof|Min Pressure	0.00	-19.27	0.00	223.16	0.00	0.00
Normal to Eave: Walls+Roof|+GCPi	0.00	8.82	-11.44	-63.11	0.00	0.00
Normal to Eave: Walls Only|+GCPi	0.00	13.15	0.00	-133.33	0.00	0.00
Normal to Eave: Walls+Roof|-GCPi	0.00	11.45	0.36	-100.99	0.00	0.00
Normal to Eave: Walls Only|-GCPi	0.00	11.31	0.00	-98.37	0.00	0.00
Normal to Eave: Walls+Roof|Min Pressure	0.00	15.80	0.00	-157.32	0.00	0.00
Parallel to Ridge: Walls+Roof|+GCPi	-4.63	0.75	-8.49	-14.19	-84.43	10.75
Parallel to Ridge: Walls Only|+GCPi	-4.63	3.96	0.00	-75.27	-45.48	-4.00
Parallel to Ridge: Walls+Roof|-GCPi	-4.63	2.26	0.36	-42.93	-45.48	-4.00
Parallel to Ridge: Walls Only|-GCPi	-4.63	2.12	0.00	-40.31	-45.48	-4.00
Parallel to Ridge: Walls+Roof|Min Pressure	-6.69	0.00	0.00	0.00	-64.52	-5.38

** Per MecaLibrary.RefEntry, Use greater of Shear calculated with or without roof.*
** X= Along Building ridge, Y = Normal to Building Ridge, Z = Vertical*
** Minimum Pressures applied to a vertical plane normal to wind.*
** Reactions calculated about the geometric center of the footprint*

Controlling lateral wind design load - Compare with Seismic forces

Components and Cladding (C&C) Zone Summary per Ch 30 Pt 1:

h/W	=	Ratio of mean roof height to building width	= 0.674
h/L	=	Ratio of mean roof height to building length	= 0.285
h	=	Mean structure height	= 14.830 ft
h_{grade}	=	Elevation from Grade to Top of Structure	= 14.830 ft
K_h	=	$2.01\bullet(15/Z_g)^{2/\alpha}$ [Tbl 26.10-1]	= 0.575
K_{zt}	=	No Topographic feature specified	= 1.000
K_d	=	Wind Directionality Factor per Tbl 26.6-1	= 0.85
$+GC_{pi}$	=	Enclosed Positive Internal Pressure Tbl 26.13-1	= +0.18
$-GC_{pi}$	=	Enclosed Negative Internal Pressure Tbl 26.13-1	= -0.18
LF	=	Load Factor based upon STRENGTH Design	= 1.00
K_e	=	Ground Elev Factor [Tbl 26.9-1]	= 0.983
q_h	=	$0.00256\bullet K_h\bullet K_{zt}\bullet K_d\bullet K_e\bullet V^2*LF$ [Eq 26.10-1]	= 11.80 psf
LHD	=	Least Horizontal Dimension: Min(B, L)	= 22.000 ft
a_1	=	$Min(0.1\bullet LHD, 0.4\bullet h)$	= 2.200 ft
a	=	$Max(a_1, 0.04\bullet LHD, 3\ ft\ [0.9\ m])$	= 3.000 ft
h/B	=	Ratio of mean roof height to least horizontal dim: h/B	= 0.674



Wind Pressure Summary for C&C Zones based Upon Areas Ch 30 Pt 1 (Table 1 of 2)
All wind pressures include a Load Factor (LF) of 1.0

Zone	Figure	Pos A ≤ 10 ft² psf	Neg A ≤ 10 ft² psf	Pos A = 20 ft² psf	Neg A = 20 ft² psf	Pos A = 50 ft² psf	Neg A = 50 ft² psf
1	30.3-5A	16.00	-16.00	16.00	-16.00	16.00	-16.00
2	30.3-5A	16.00	-17.47	16.00	-17.11	16.00	-16.64
2'	30.3-5A	16.00	-21.01	16.00	-20.65	16.00	-20.18
3	30.3-5A	16.00	-23.37	16.00	-21.24	16.00	-18.42
3'	30.3-5A	16.00	-32.81	16.00	-29.26	16.00	-24.56
4	30.3-1	16.00	-16.00	16.00	-16.00	16.00	-16.00
5	30.3-1	16.00	-17.00	16.00	-16.00	16.00	-16.00

Wind Pressure Summary for C&C Zones based Upon Areas Ch 30 Pt 1 (Table 2 of 2)
All wind pressures include a Load Factor (LF) of 1.0

Zone	Figure	Pos A = 100 ft² psf	Neg A = 100 ft² psf	Pos A = 200 ft² psf	Neg A = 200 ft² psf	Pos A > 500 ft² psf	Neg A > 500 ft² psf
1	30.3-5A	16.00	-16.00	16.00	-16.00	16.00	-16.00
2	30.3-5A	16.00	-16.29	16.00	-16.29	16.00	-16.29
2'	30.3-5A	16.00	-19.83	16.00	-19.83	16.00	-19.83
3	30.3-5A	16.00	-16.29	16.00	-16.29	16.00	-16.29
3'	30.3-5A	16.00	-21.01	16.00	-21.01	16.00	-21.01
4	30.3-1	16.00	-16.00	16.00	-16.00	16.00	-16.00
5	30.3-1	16.00	-16.00	16.00	-16.00	16.00	-16.00

** A is effective wind area for C&C: Span Length * Effective Width*
** Effective width need not be less than 1/3 of the span length*
** Maximum and minimum values of pressure shown.*
** + Pressures acting toward surface, - Pressures acting away from surface*
** Per § 30.2.2 the Minimum Pressure for C&C is 16.00 psf [0.766 kPa] {Includes LF}*
** Interpolation can be used for values of A that are between those values shown.*

Calculation of Seismic Response Coefficient, Cs

(ASCE 7-16, Chapter 11 and 12, Equivalent Lateral Force "ELF" Procedure)

BUILDING INFORMATION :

Risk Category :	II	(ASCE 7-16 Table 1.5-1 & IBC Table 1604.5)
Importance Factor, Ie :	1.00	(ASCE 7-16 Table 1.5-2)
Response Modification Factor, R :	6.5	(ASCE 7-16 Table 12.2-1)
Overstrength Factor, Ω :	3	(ASCE 7-16 Table 12.2-1)
Deflection Amplification Factor, Cd :	4	(ASCE 7-16 Table 12.2-1)

SITE INFO & SEISMIC ACCELERATIONS :

Site Class :	D (default)	(IBC Section 1613.2.2, "D" Assumed or per Geotech.)
Ss :	1.257	
S1 :	0.434	
Fv:	1.866	(ASCE 7-16 TABLE 11.4-2)
Sds :	1.006	
Sd1 :	0.540	(Eqn 11.4-2 & 11.4-4)
Seismic Design Category :	D	(ASCE 7-16 TABLE 11.6-1 & 11.6-2)

PERIOD DETERMINATION :

Ct :	0.02	(ASCE 7-16 Table 12.8-2)
x :	0.75	(ASCE 7-16 Table 12.8-2)
hn (ft) :	14.67	
Ta = Ct*hn^x :	0.150	(Eqn 12.8-7)
Ts = (Sd1/Sds) :	0.537	(ASCE 7-16 11.4.6)
1.5*Ts :	0.805	

CALCULATE Cs :

Cs = Sds/(R/I) :	0.155	(Eqn 12.8-2)
Max Cs = Sd1/(Ta*(R/I)) :	0.554	(Eqn 12.8-3)
Min Cs = 0.044*Sds*I > 0.01 :	0.044	(Eqn 12.8-5)
Min Cs = 0.5*S1/(R/I) :	0.000	(Eqn 12.8-6, for S1 > 0.6g)
Minimum Cs :	0.044	
Cs :	**0.155**	
Base Shear, V = Cs * W :	0.155 * W	

SITE CLASS CHECKS :

Check ASCE-16, 11.4.8, Site Class F :	Site Response Analysis Not Required	Ground Motion Hazard Analysis <u>is</u> Required for seismically isolated structures or structures with damping systems on sites with S1 >/= 0.6
Check ASCE-16, 11.4.8, Site Class E :	Ground Motion Hazard Analysis Not Required	
Check ASCE-16, 11.4.8, Site Class D, Exception 2:	Ground Motion Hazard Analysis Not Required	

BSE
B rienen ***S*** tructural ***E*** ngineers, P.S.

Seismic Weight

-Roof

Metal Roofing	1.5 psf
Wood Sheathing	2.2 psf
Metal Truss Framing	5.0 psf
Insulation	2.5 psf
Mech.	1.5 psf
Misc.	1.8 psf
Total	13 psf

-Exterior Walls

Metal Siding	1.5 psf
Wood Sheathing	1.5 psf
Gyp Board	2.8 psf
Insulation	2.2 psf
Metal Stud Framing	2.0 psf
Total	10 psf

Seismic Base Shear

Roof:
(52ft x 22ft) x (13psf) = 14872lbs

Exterior Walls:
perimeter = (2 x 52ft) + (2 x 22ft) = 148ft
wall height = 14.67ft
(148ft x 14.67ft/2) x (10psf) = 10856lbs

Seismic Weight = 14872lbs + 10974lbs = 25846lbs

Base Shear, V = Cs * W = 0.155 * 25846lbs = 4006lbs = 4.0kips

Compare with Wind Base Shear
-
Wind Controls, V = 19.27kips
(Normal to ridge)
V = 6.69kips
(Parallel to ridge)

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Diaphragm Design

-Diphragm Forces

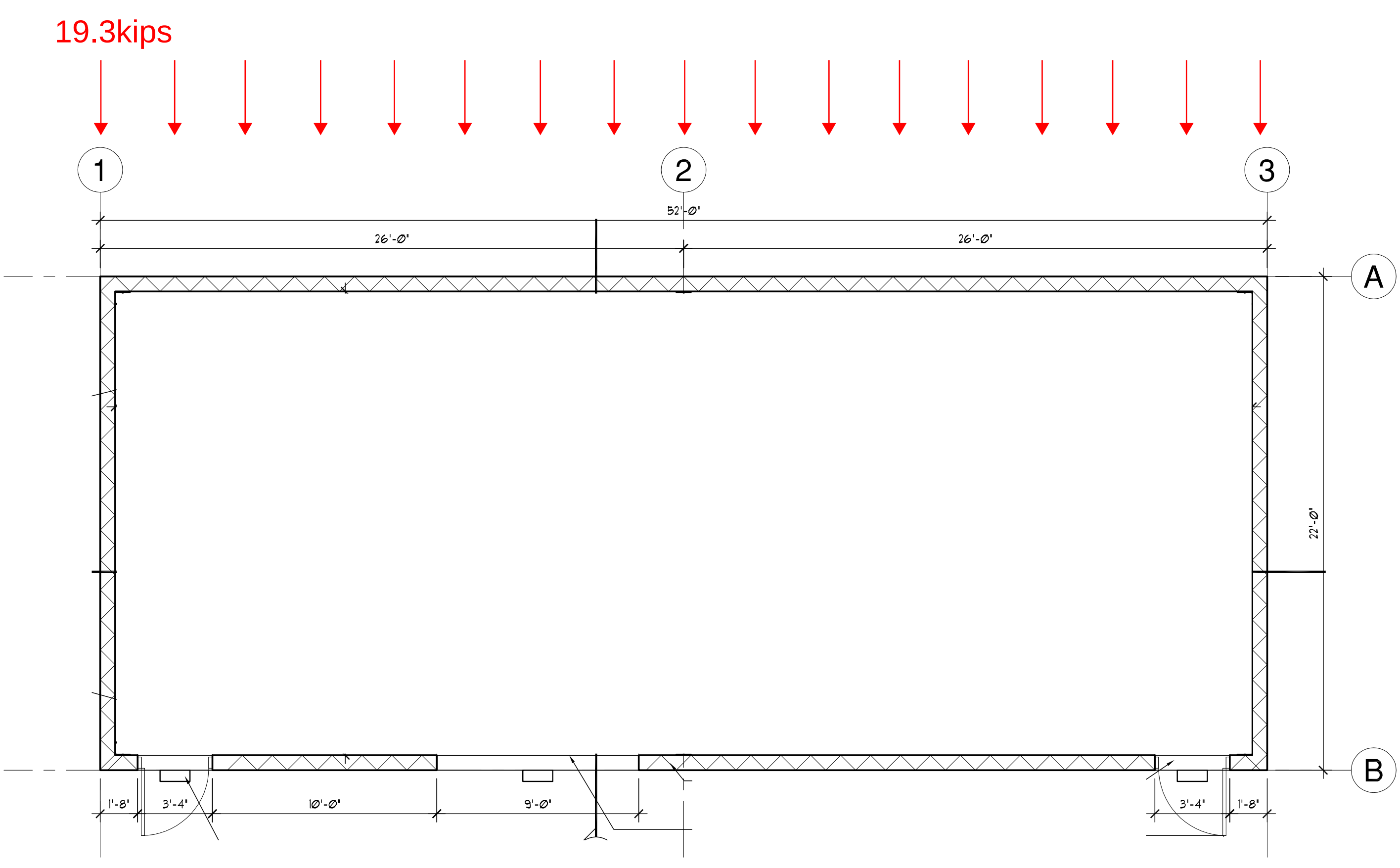
Normal to Ridge:
V = 19.3kips (LRFD)
= 0.6*19.3kips = 11.6kips (ASD)

Distributed Wind Load w = 11.6kips / 52ft = 223lbs /ft

- 2 walls ea end of diaphragm
Force to each wall = (11.6kips) / 2 = 5.8kips ea

-Max Diaphragm shear @ gridlines 1 & 2
v = (5.8kips) / 22ft = 0.264kips/ft = 264 lbs/ft

-Max chord Forces @ gridlines A & B
Mmax = (223lbs/ft) * (52ft)^2 / 8 = 89232 lb-ft
Total Chord Force, T/C = (89232lb-ft) / 22ft = 4056lbs
Linear chord force = 4056lbs / 52ft = 78lbs/ft



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Diaphragm Design

-Diphragm Forces

Parallel to Ridge:
V = 6.7kips (LRFD)
= 0.6*6.7kips = 4.0kips (ASD)

Distributed Wind Load w = 4.0kips / 22ft = 183lbs/ft

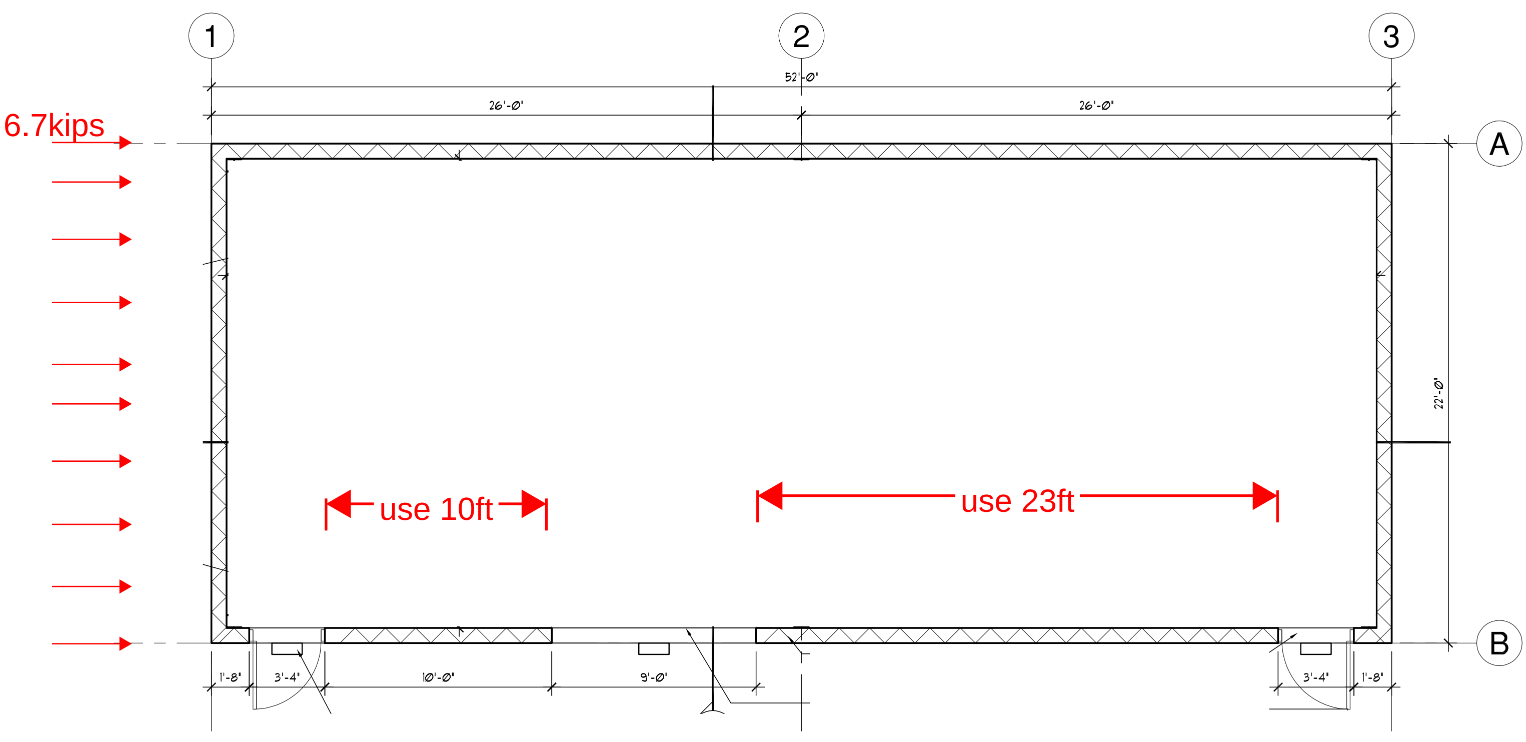
- 2 walls ea end of diaphragm

Force to each wall = (4.0kips) / 2 =2.0kips ea

-Max Diaphragm shear @ gridline B
v = (2.0kips) / 52ft = 0.039kips/ft = 39 lbs/ft

-Max Diaphragm shear @ gridline A
v = (2.0kips) / (10ft + 23ft) = 0.061kips/ft = 61 lbs/ft

-Max chord Forces @ gridlines 1&2
Mmax = (61lbs/ft) * (22ft)^2 / 8 = 3691 lb-ft
Total Chord Force, T/C = (3691lb-ft) /52ft = 71lbs
Linear chord force = 71lbs / (9ft+24ft) = 2lbs/ft



BSE

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Diaphragm Design

-Diphragm Forces

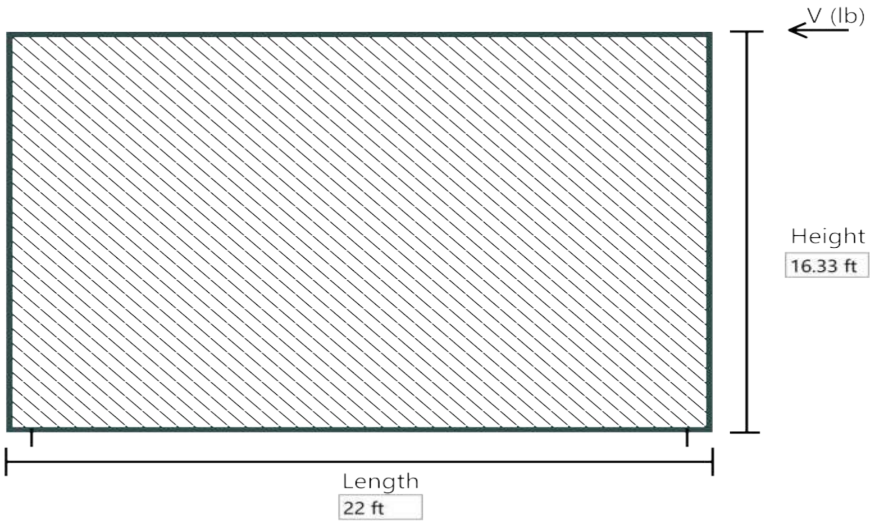
Table F2.4-1
Nominal Shear Strength (v_n) per Unit Length for Diaphragms Sheathed With Wood Structural Panel Sheathing [1, 2]
***United States and Mexico* (lb/ft)**

Sheathing	Thickness (in.)	Blocked				Unblocked	
		Screw spacing at diaphragm boundary edges and at all continuous panel edges (in.)				Screws spaced maximum of 6 in. on all supported edges	
		6	4	2.5	2	Load perpendicular to unblocked edges and continuous panel joints	All other configurations
		Screw spacing at all other panel edges (in.)					
		6	6	4	3		
Structural I	3/8	768	1022	1660	2045	685	510
	7/16	768	1127	1800	2255	755	565
	15/32	925	1232	1970	2465	825	615
C-D, C-C and other graded wood structural panels	3/8	690	920	1470	1840	615	460
	7/16	760	1015	1620	2030	680	505
	15/32	832	1110	1770	2215	740	555

1. For SI: 1" = 25.4 mm, 1 ft = 0.305 m, 1 lb = 4.45 N
2. For *diaphragms* sheathed with *wood structural panels*, tabulated R_n values are applicable for short-term *load* duration (seismic *loads*).

Using 15/32" min thickness OSB @ roof w/ #8 SMS @ 6"oc at panel edges-
825lb/ft / W = 330plf > 264lb/ft [OK]

Simple Shear Wall design is intended for use with element and component design or other designs where special seismic considerations are not required. For design of building LFRS shear walls, use CFS Designer's LFRS Shear Wall design module.



Chord Data

Chords: 600S162-54 (50) Back-To-Back
AISI S100-16

Chord Fastener Spacing, a =	12 in
Shearwall Chord Force =	4353 lb
Includes Anchor Offset =	2.0 in
Additional Axial Loads =	1000 lb
Total Axial Loads =	5353 lb
KyLy, KtLt for Axial Capacity =	Sheathed
Maximum KL/r =	86
Allowable Axial Load =	6444 lb
Input Chord Moment =	0 ft-lb
Flexural Bracing =	Full
Distortional Buckling Inputs for Moment and Axial	
K-phi =	0 lb-in/in
Lm =	None
Allowable Moment =	3860 ft-lb
Chord Interaction =	0.831

Overturning Uplift Data

Anchor offset Each End =	2.0 in
Uplift at Anchor - Wind =	4353 lb
Uplift at Anchor - Seismic =	1047 lb

Holdown Data

Holdown: S/HDU11 - 54
Tension Force: 4353 lb
Allowable Tension: 7665 lb
Interaction: 0.57

Load Data (Factored ASD)

V(wind) =	5820 lb
V(seismic) =	1400 lb

Sheathing Data

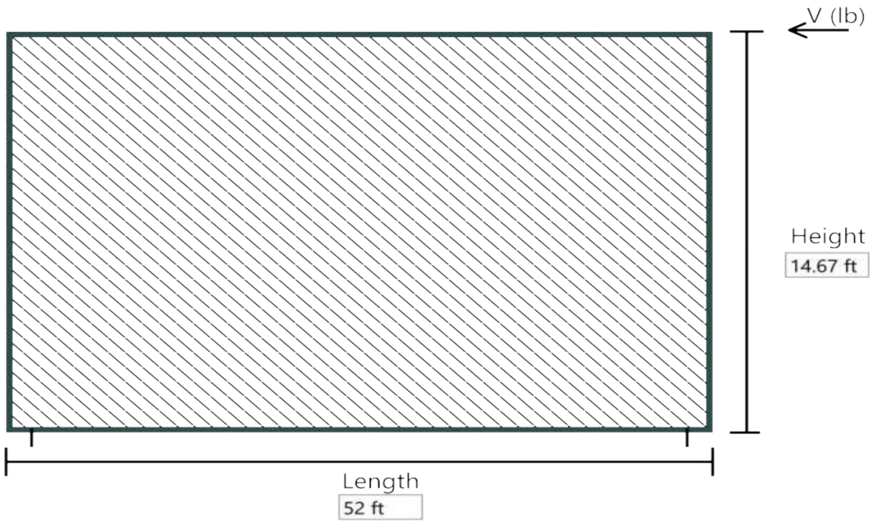
Shear values per IBC 2018 (AISI S240-15)
See AISI S240-15 for additional information

Stud Thickness = 54 mils
Sheathing: 7/16 Rated Sheathing (OSB) 1 side
Fasteners: 6-inches oc edges, 12-inches oc field

Screws attaching panels to CFS steel framing shall comply with ASTM C1513. Also, for framing members that are 54 mil and thinner, use minimum #8. For 68 mil and thicker, use min. #10.

Aspect Ratio =	0.74:1
Allowable Aspect Ratio for Seismic =	2.00:1
Allowable Aspect Ratio for Wind =	2.00:1
Unit Shear (Wind) =	265 lb/ft
Allowable Unit Shear (Wind) =	455 lb/ft
Unit Shear (Seismic) =	64 lb/ft
Allowable Unit Shear (Seismic) =	455 lb/ft

Simple Shear Wall design is intended for use with element and component design or other designs where special seismic considerations are not required. For design of building LFRS shear walls, use CFS Designer's LFRS Shear Wall design module.



Chord Data

Chords: 600S162-54 (50) Back-To-Back
AISI S100-16

Chord Fastener Spacing, a =	12 in
Shearwall Chord Force =	577 lb
Includes Anchor Offset =	2.0 in
Additional Axial Loads =	0 lb
Total Axial Loads =	577 lb
KyLy, KtLt for Axial Capacity =	Sheathed
Maximum KL/r =	78
Allowable Axial Load =	6444 lb
Input Chord Moment =	0 ft-lb
Flexural Bracing =	Full
Distortional Buckling Inputs for Moment and Axial	
K-phi =	0 lb-in/in
Lm =	None
Allowable Moment =	3860 ft-lb
Chord Interaction =	0.090

Overturning Uplift Data

Anchor offset Each End =	2.0 in
Uplift at Anchor - Wind =	577 lb
Uplift at Anchor - Seismic =	396 lb

Holdown Data

Holdown: S/HDU4 - 54

Tension Force: 577 lb

Allowable Tension: 2550 lb

Interaction: 0.23

Load Data **(Factored ASD)**

V(wind) =	2040 lb
V(seismic) =	1400 lb

Sheathing Data

Shear values per IBC 2018 (AISI S240-15)

See AISI S240-15 for additional information

Stud Thickness = 54 mils

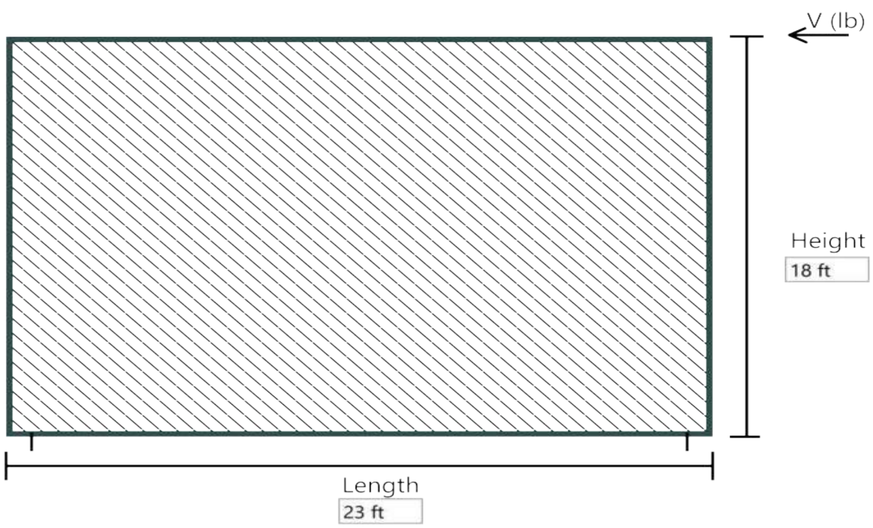
Sheathing: 7/16 Rated Sheathing (OSB) 1 side

Fasteners: 6-inches oc edges, 12-inches oc field

Screws attaching panels to CFS steel framing shall comply with ASTM C1513. Also, for framing members that are 54 mil and thinner, use minimum #8. For 68 mil and thicker, use min. #10.

Aspect Ratio =	0.28:1
Allowable Aspect Ratio for Seismic =	2.00:1
Allowable Aspect Ratio for Wind =	2.00:1
Unit Shear (Wind) =	39 lb/ft
Allowable Unit Shear (Wind) =	455 lb/ft
Unit Shear (Seismic) =	27 lb/ft
Allowable Unit Shear (Seismic) =	455 lb/ft

Simple Shear Wall design is intended for use with element and component design or other designs where special seismic considerations are not required. For design of building LFRS shear walls, use CFS Designer's LFRS Shear Wall design module.



Chord Data

Chords: 600S162-54 (50) Single
AISI S100-16

Shearwall Chord Force =	1170 lb
Includes Anchor Offset =	2.0 in
Additional Axial Loads =	0 lb
Total Axial Loads =	1170 lb
KyLy, KtLt for Axial Capacity =	Sheathed
Maximum KL/r =	95
Allowable Axial Load =	3222 lb
Input Chord Moment =	0 ft-lb
Flexural Bracing =	Full
Distortional Buckling Inputs for Moment and Axial	
K-phi =	0 lb-in/in
Lm =	None
Allowable Moment =	3860 ft-lb
Chord Interaction =	0.363

Overturning Uplift Data

Anchor offset Each End =	2.0 in
Uplift at Anchor - Wind =	1170 lb
Uplift at Anchor - Seismic =	803 lb

Holdown Data

Holdown: S/HDU4 - 54

Tension Force: 1170 lb

Allowable Tension: 2550 lb

Interaction: 0.46

Load Data **(Factored ASD)**

V(wind) =	1484 lb
V(seismic) =	1018 lb

Sheathing Data

Shear values per IBC 2018 (AISI S240-15)

See AISI S240-15 for additional information

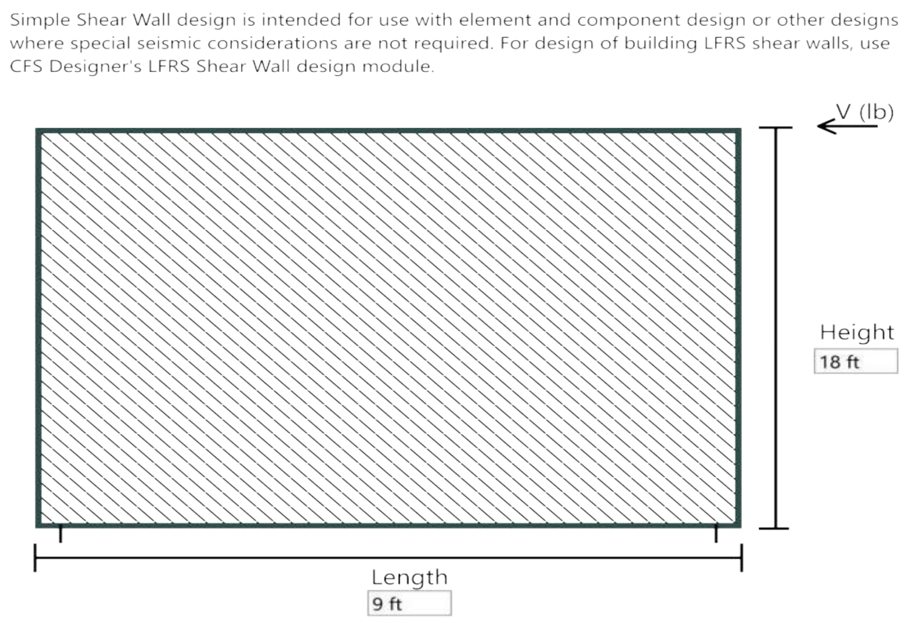
Stud Thickness = 54 mils

Sheathing: 7/16 Rated Sheathing (OSB) 1 side

Fasteners: 6-inches oc edges, 12-inches oc field

Screws attaching panels to CFS steel framing shall comply with ASTM C1513. Also, for framing members that are 54 mil and thinner, use minimum #8. For 68 mil and thicker, use min. #10.

Aspect Ratio =	0.78:1
Allowable Aspect Ratio for Seismic =	2.00:1
Allowable Aspect Ratio for Wind =	2.00:1
Unit Shear (Wind) =	65 lb/ft
Allowable Unit Shear (Wind) =	455 lb/ft
Unit Shear (Seismic) =	44 lb/ft
Allowable Unit Shear (Seismic) =	455 lb/ft



Chord Data

Chords: 600S162-54 (50) Single
AISI S100-16

Shearwall Chord Force =	1133 lb
Includes Anchor Offset =	2.0 in
Additional Axial Loads =	0 lb
Total Axial Loads =	1133 lb
KyLy, KtLt for Axial Capacity =	Sheathed
Maximum KL/r =	95
Allowable Axial Load =	3222 lb
Input Chord Moment =	0 ft-lb
Flexural Bracing =	Full
Distortional Buckling Inputs for Moment and Axial	
K-phi =	0 lb-in/in
Lm =	None
Allowable Moment =	3860 ft-lb
Chord Interaction =	0.352

Overturning Uplift Data

Anchor offset Each End =	2.0 in
Uplift at Anchor - Wind =	1133 lb
Uplift at Anchor - Seismic =	776 lb

Holdown Data

Holdown: S/HDU4 - 54
Tension Force: 1133 lb
Allowable Tension: 2550 lb
Interaction: 0.44

Load Data (Factored ASD)

V(wind) =	556 lb
V(seismic) =	381 lb

Sheathing Data

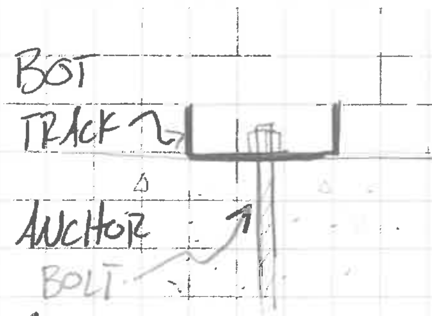
Shear values per IBC 2018 (AISI S240-15)
See AISI S240-15 for additional information

Stud Thickness = 54 mils
Sheathing: 7/16 Rated Sheathing (OSB) 1 side
Fasteners: 6-inches oc edges, 12-inches oc field

Screws attaching panels to CFS steel framing shall comply with ASTM C1513. Also, for framing members that are 54 mil and thinner, use minimum #8. For 68 mil and thicker, use min. #10.

Aspect Ratio =	2.00:1
Allowable Aspect Ratio for Seismic =	2.00:1
Allowable Aspect Ratio for Wind =	2.00:1
Unit Shear (Wind) =	62 lb/ft
Allowable Unit Shear (Wind) =	455 lb/ft
Unit Shear (Seismic) =	42 lb/ft
Allowable Unit Shear (Seismic) =	455 lb/ft

Shear Transfer - Bot Track Anchors



AISI S100 - Section E3

Track thickness, $t = 54$ mil

$F_y = 50$ ksi ; $F_u = 65$ ksi

Anchor Bolt ϕ, $D = 0.625"$

$A_b = 0.31 in^2$

AISI S100 E3.3.1.1 $\quad P_n = C \times m_f \times d \times t \times F_u \qquad \Omega = 2.5$

$C = 4 - 0.1(d/t) = 4 - 0.1(0.625"/0.054") = 2.84$

$m_f = 0.75$ (w/ washer)

$P_n = (2.84)(0.75)(0.625")(.054")(65 ksi)$

$= 4.67$ kips

$P_n/\Omega = 4.67 \text{ kips} / 2.5 = 1.87$ kips

Try Bolt Hole Deformation

AISI S100 E3.3.2.1 $\quad P_n = (4.64\alpha t + 1.53) d \times t \times F_u \quad ; \quad \Omega = 2.22$

$\alpha = 1.0$

$P_n = (4.64 \cdot (1.0) \cdot (.054") + 1.53) \cdot (0.625")(.054")\cdot(65 ksi)$

$= 3.91$ kips

$P_n/\Omega = 3.91 \text{ kips} / 2.22 = 1.76$ kips per anchor

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Alt Bot Track Anchor	Date:	2/14/2024
Fastening point:			

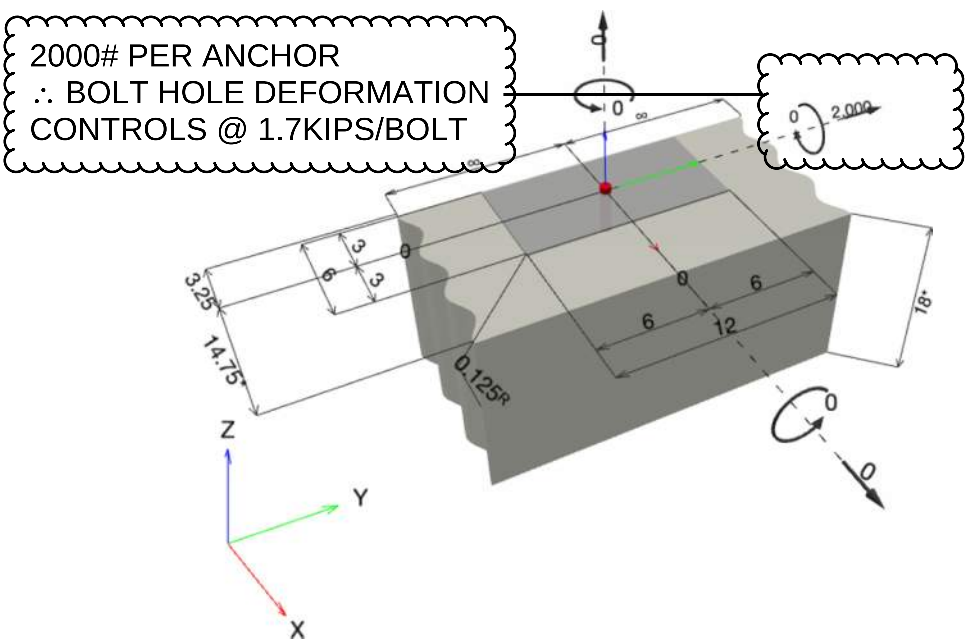
Specifier's comments:

1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) 5/8 (3 1/4)**
Item number:	418078 KH-EZ 5/8"x3 1/2"
Effective embedment depth:	$h_{ef,act}$ = 2.390 in., h_{nom} = 3.250 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	4/1/2022 | 12/1/2023
Proof:	Design Method ACI 318-19 / Mech
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.125 in.
Anchor plate^R :	l_x x l_y x t = 6.000 in. x 12.000 in. x 0.125 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 2500, f_c' = 2,500 psi; h = 18.000 in.
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar

^{R} - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



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Phone I Fax:	|	E-Mail:	
Design:	Alt Bot Track Anchor	Date:	2/14/2024
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V_x = 0; V_y = 2,000; M_x = 0; M_y = 0; M_z = 0;	no	99

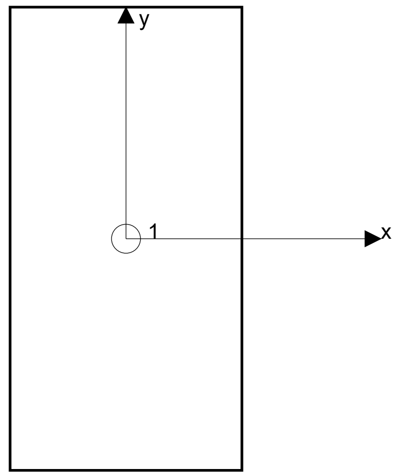
2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	2,000	0	2,000

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]



Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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Design:	Alt Bot Track Anchor	Date:	2/14/2024
Fastening point:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	2,000	6,732	30	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	2,000	2,037	99	OK
Concrete edge failure in direction x-**	2,000	2,968	68	OK

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

V_{sa} = ESR value	refer to ICC-ES ESR-3027
$\phi\ V_{steel} \geq V_{ua}$	ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.27	90,180

Calculations

V_{sa} [lb]
11,220

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
11,220	0.600	6,732	2,000

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Phone I Fax:	|	E-Mail:	
Design:	Alt Bot Track Anchor	Date:	2/14/2024
Fastening point:			

4.2 Pryout Strength

$V_{cp} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b\right]$ ACI 318-19 Eq. (17.7.3.1a)

$\phi\ V_{cp} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
1	2.390	3.250	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
3.630	17	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
49.01	51.41	0.972	1.000	3,141

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi\ V_{cp}$ [lb]	V_{ua} [lb]
2,910	0.700	2,037	2,000

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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Alt Bot Track Anchor	Date:	2/14/2024
Fastening point:			

4.3 Concrete edge failure in direction x-

$V_{cb} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ — ACI 318-19 Eq. (17.7.2.1a)

$\phi\ V_{cb} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)

$A_{Vc0} = 4.5\ c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c}\ c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1a)

Variables

c_{a1} [in.]	c_{a2} [in.]	$\psi_{c,V}$	h_a [in.]	l_e [in.]
3.250	-	1.000	18.000	2.390

λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
1.000	0.625	2,500	2.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
47.53	47.53	1.000	1.000	2,120

Results

V_{cb} [lb]	$\phi_{concrete}$	$\phi\ V_{cb}$ [lb]	V_{ua} [lb]
4,240	0.700	2,968	2,000

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-19, Section 26.7.

Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Alt Bot Track Anchor	Date:	2/14/2024
Fastening point:			

Fastening meets the design criteria!

www.hilti.com

Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Alt Bot Track Anchor	Date:	2/14/2024
Fastening point:			

6 Installation data

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) 5/8 (3 1/4)

Item number: 418078 KH-EZ 5/8"x3 1/2"

Maximum installation torque: 1,020 in.lb

Hole diameter in the base material: 0.625 in.

Hole depth in the base material: 3.625 in.

Minimum thickness of the base material: 5.000 in.

Profile: no profile

Hole diameter in the fixture: d_f = 0.750 in.

Plate thickness (input): 0.125 in.

Recommended plate thickness: not calculated

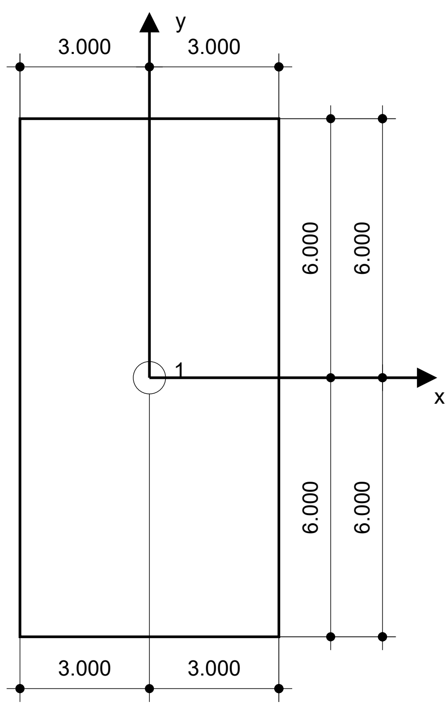
Drilling method: Hammer drilled

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Hilti KH-EZ screw anchor with 3.25 in embedment, 5/8 (3 1/4), Carbon steel, installation per ESR-3027

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque wrench • Hilti SIW 9-A22 Impact Wrench



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	3.250	14.750	-	-

Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Alt Bot Track Anchor	Date:	2/14/2024
Fastening point:			

7 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

Vertical Design

BSE

B rienen **S** tructural **E** ngineers, P.S.

Building Weights

-Roof

Wood Sheathing	2.2 psf
Metal Joist Framing	2.8 psf
Insulation	2.5 psf
Gypsum Sheathing	2.2 psf
Mech.	1.5 psf
Misc.	1.8 psf
Total Dead Load	13 psf
Total Live Load	20 psf
Total Snow Load	25 psf

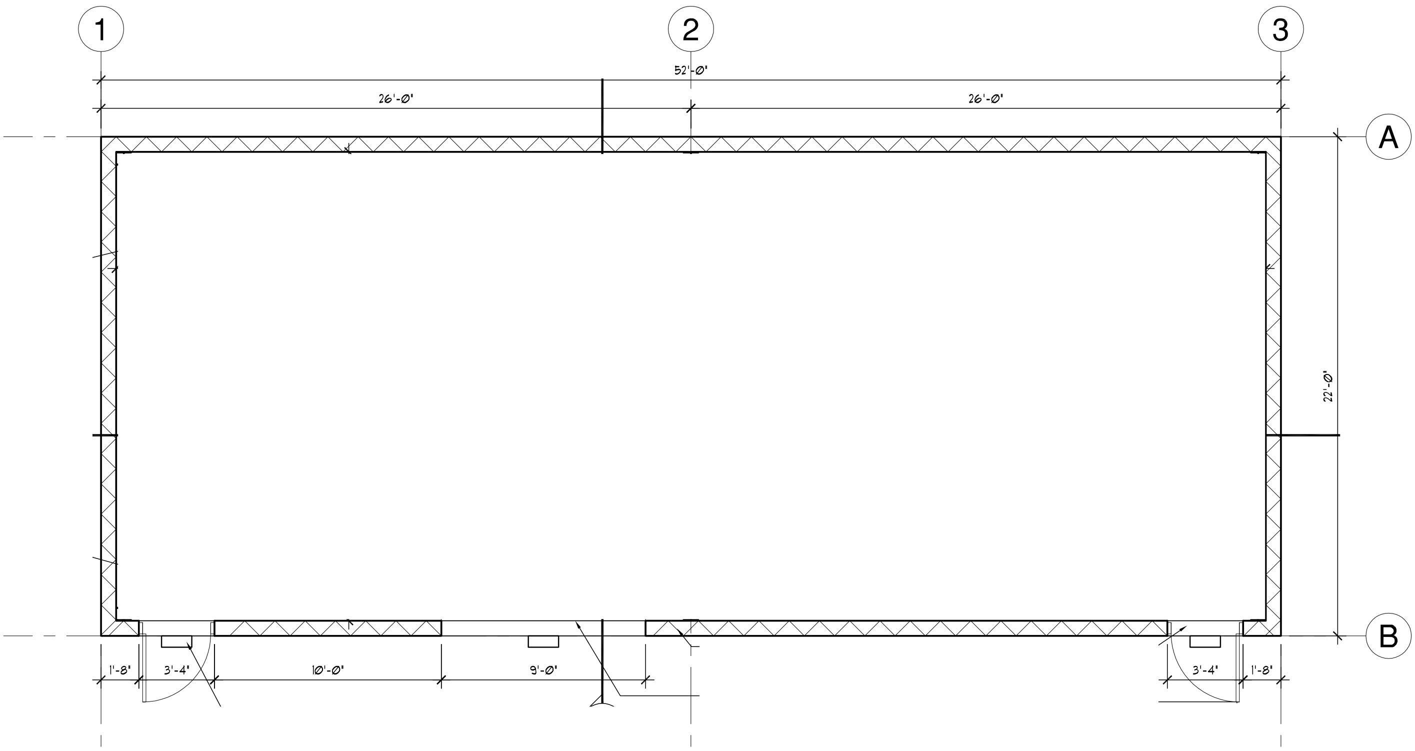
-Exterior Walls

Metal Siding	1.5 psf
Wood Sheathing	1.5 psf
Gyp Board	2.8 psf
Insulation	2.2 psf
Metal Stud Framing	2.0 psf
Total Dead Load	10 psf
Lateral Live Load	5 psf
Wind Load	16 psf

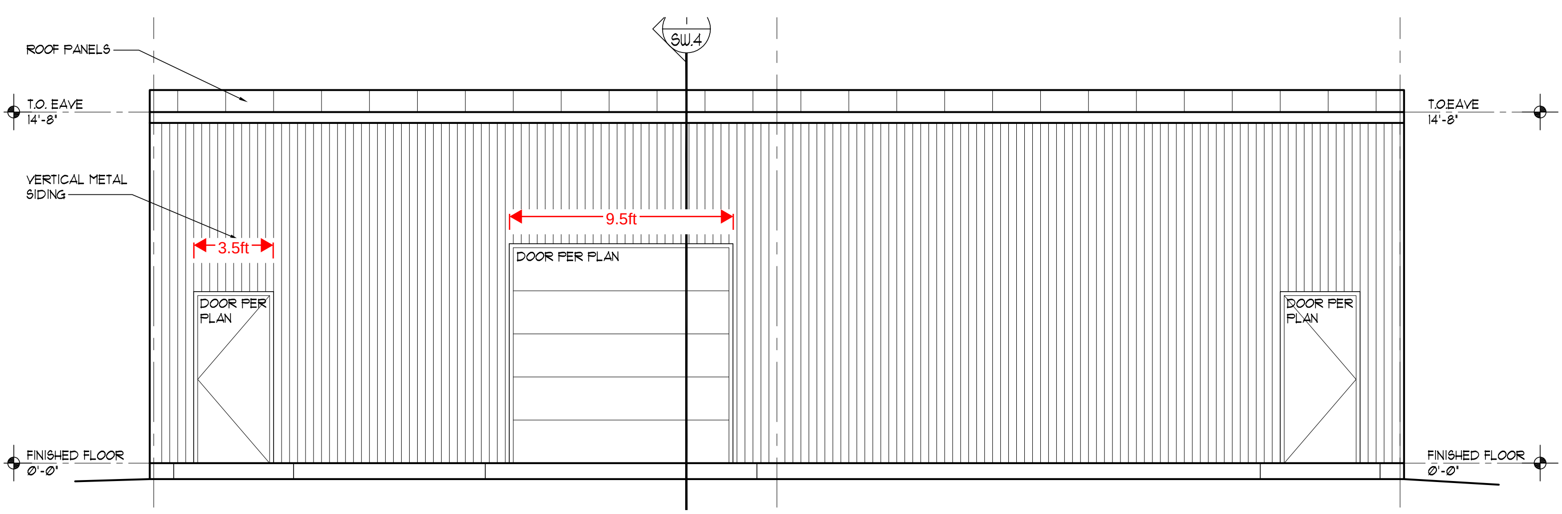
BSE

B rienen ***S*** tructural ***E*** ngineers, P.S.

Building Plan



Building Elevation



WIND LOAD - ASCE 7-16

98 mph, Exposure B, Mean Roof Height = 16.5 ft

K_{zt} at Base = 1

K_d = 0.85 , Roof Slope 9.46 degrees (2:12)

Enclosed Building, GC_{pi} = 0.18

(Wind Loads Shown are for Alternate Basic Load Combinations Using Allowable Stress Design and are Multiplied by a Factor of 0.6 to convert to ASD)

WALL COMPONENTS AND CLADDING per ASCE7-16 Figure 30.3-1

Tributary Area (ft2)	GCp by Zone Zone 4 (+/-)	Zone 5 (+/-)
10 ft^2	0.90/-0.99	0.90/-1.26
50 ft^2	0.79/-0.88	0.79/-1.04
500 ft^2	0.63/-0.72	0.63/-0.72

Height z (ft)	K_z	K_{zt}	K_e	q_z (psf)	Tributary Area (ft2)	Wind Pressures (psf) by Zone () **Windward (4,5)**	**Leeward (4)**	**Leeward (5)**
0 - 16.5	0.70	1.00	1.00	14.64	10	9.6	-10.3	-12.6
					50	9.6	-9.6	-10.7
					500	9.6	-9.6	-9.6

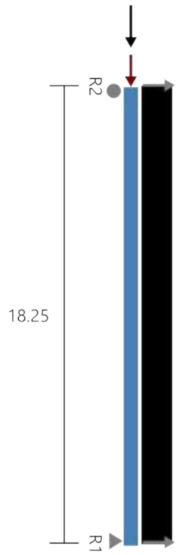
ROOF COMPONENTS AND CLADDING - MONOSLOPE ROOF ASCE7-16 Figure 30.3-5A

K_h = 0.70; K_{zt} at roof = 1.00; K_e = 1.00; q_h = 14.64 psf

	Positive Pressure, p (psf)				Negative Pressure, p (psf)			
	A=10		A=100		A=10		A=100	
Zone	GC_p	p	GC_p	p	GC_p	p	GC_p	p
1	0.30	9.60	0.20	9.60	-1.10	-11.24	-1.10	-11.24
2	0.30	9.60	0.20	9.60	-1.30	-13.00	-1.20	-12.12
3	0.30	9.60	0.20	9.60	-1.80	-17.39	-1.20	-12.12
2'	0.30	9.60	0.20	9.60	-1.60	-15.64	-1.50	-14.76
3'	0.30	9.60	0.20	9.60	-2.60	-24.42	-1.60	-15.64

Project Name: Centeris
Model: 18'-3" Bearing Wall
Code: AISI S100-16

Date: 03/08/2024
Simpson Strong-Tie® CFS Designer™ 5.2.3.0



Section :	600S162-54 (50 ksi) @ 24" o.c. Single C Stud (punched)		
Maxo = 2313.4 ft-lb	**Va =** 2822.9 lb	**I =** 2.86 in^4	

Wind Selection: C&C Wind, Leeward (5)
Tributary Area: Span: Length^2/3
Loads have not been modified for strength checks
Loads have been multiplied by 0.70 for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	48.0", 48.0"	48.0", 219.0"	LSUBH3.25 (Min)	0.36

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R2	-177.03	1.00	598.9	0.0	0.15	NO
R1	-177.03	1.00	598.9	0.0	0.15	NO

"*" after support means punched near support

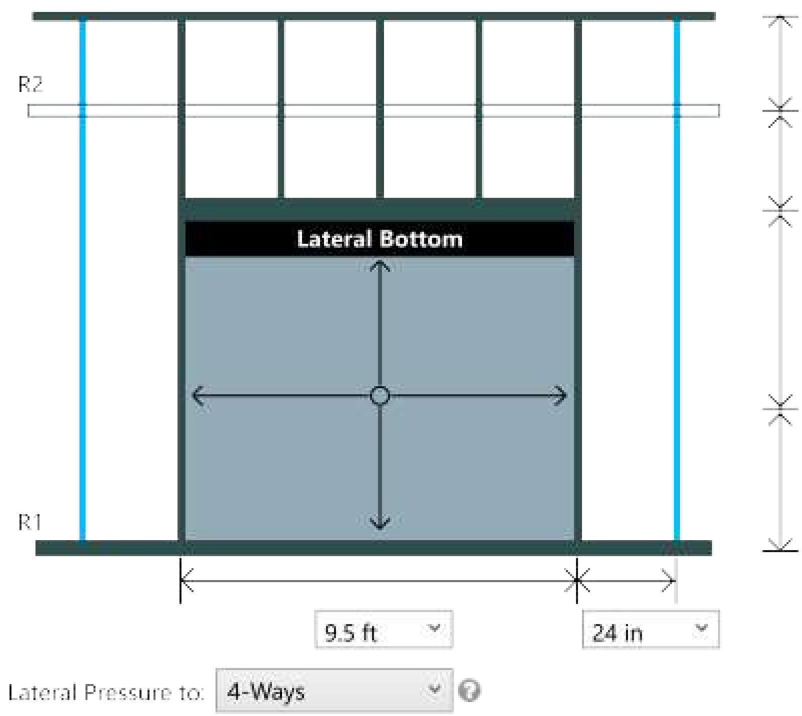
Gravity Load

Type	Load (lb)
Uniform	20.00plf (Span)
P1y	857.00lb @ 18.25ft

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	1222.0(c)	4458.1(c)	27%	KΦ=0.00 lb-in/in Max KL/r = 97
	Max. Shear, lbs	177.0	1947.4	9%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	807.7	1930.2	42%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	807.7	2079.7	39%	
	Shear/Moment	0.35	1.00	35%	Shear 0.0, Moment 807.7
	Axial/Moment	0.70	1.00	70%	Axial 1049.4(c), Moment 805.3
	Deflection Span, in	0.402	--meets L/545--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R2	-177.0	0.0	By Others & Anchorage Designed by Engineer	NA	NA
R1	-177.0	1222.0	By Others & Anchorage Designed by Engineer	NA	NA

* Reference catalog for connector and anchor requirement notes as well as screw placement requirements



Design Loads

Wind Selection :	C&C Wind, Leeward (5)
Trib. Area : Span :	Length^2/3
Wall Lateral Pressure :	-9.7 psf
Parapet Lateral Pressure :	
RO Lateral Pressure :	4-Ways
Lateral element force multiplier	
Strength :	1.0
Deflection :	0.7
Header:	Box (lateral top, bottom)
Gravity Load at Header:	10 psf
Additional Pt. Load ea. Stud :	220 lbs
Additional Jamb Axial Load :	865 lbs

Back-to-Back Member L/6 Interconnection Spacing per AISI S100 I1.1

Member	Span	Cantilever
Jamb Studs	36.5 in	0.0 in

See AISI S100 I1.1 for Add'nl Requirements

Brace Settings

Component(s)	Members(s)	Flexural Bracing	Axial KyLy	Axial KtLt	Distortional K-Phi(lb-in/in)	Distortional Lm	Interconnection Spacing
Jamb Studs	600S162-54(50), Back-To-Back	Full	48 in	48 in	0	None	12 in
Vertical Header	800S200-43(33), Boxed	Full	N/A	N/A	0	None	N/A
Lat. Top Head	600T125-54(50), Single	Full	N/A	N/A	0	None	N/A
Lat. Bottom Head	600T125-54(50), Single	Full	N/A	N/A	0	None	N/A

Analysis Results

Component(s)	Members(s)	Axial Load (lb)	Max KL/r	Max. Moment (ft-lb)	Max. Shear (lb)	Bottom Reaction (lb)	Top or End Reaction (lb)
Jamb Studs	600S162-54(50), Back-To-Back	1874.4	97	2286.8	402.3	-508.9	-272.8
Vertical Header	800S200-43(33), Boxed	N/A	N/A	2397.3	1009.4	N/A	1009.4
Lat. Top Head	600T125-54(50), Single	N/A	N/A	560.8	236.1	N/A	236.1
Lat. Bottom Head	600T125-54(50), Single	N/A	N/A	334.2	106.7	N/A	106.7

Design Results

Component(s)	Members(s)	Deflection Span	Deflection Parapet	A + M Interaction	V + M Interaction	Web Stiffners	Design OK
Jamb Studs	600S162-54(50), Back-To-Back	L/447	L/0	0.836	0.50	No	Yes
Vertical Header	800S200-43(33), Boxed	L/916	NA	0.56	0.56	R1, R2	Yes
Lat. Top Head	600T125-54(50), Single	L/1182	NA	0.38	0.38	No	Yes
Lat. Bottom Head	600T125-54(50), Single	L/2056	NA	0.23	0.04	No	Yes

Simpson Strong-Tie® Connectors @ Jambs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	-272.81	0.00	By Others & Anchorage Designed by Engineer	NA	NA

R1	-508.95	2136.88	600T125-54 (50) & (2) .157", 1" embed SST PDPA/PDPAT to 4000 nw concrete	45.02 %	82.09 %

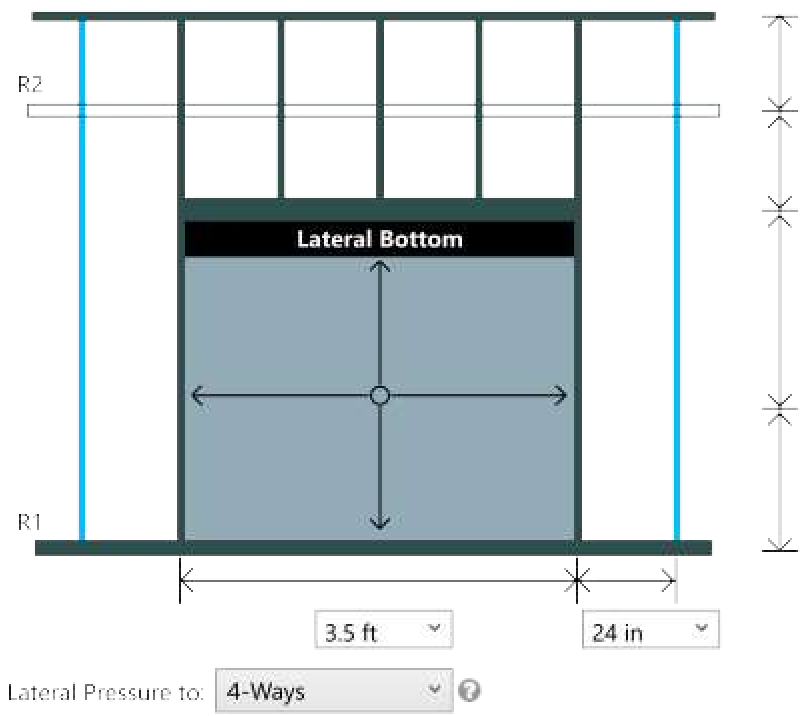
* Reference catalog for connector and anchor requirement notes as well as screw placements requirement

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Jambs

Span/Parapet	Bracing Length(in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min)[1]	LSUBH (Max)[1]	SUBH (Min)[1]	SUBH (Max)[1]	MSUBH (Min)[1]	MSUBH (Max)[1]
Span	Varies	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes:

1) Values in parentheses are stress ratios.
2) Bridging connectors are not designed for back-back, box, or built-up sections.
3) Reference www.strongtie.com for latest load data, important information, and general notes.
4) CFS Designer will not select bridging connectors unless all flexural and axial bracing settings are the same.
5) If the bracing length is larger than the span length, bridging connectors are not designed.



Design Loads

Wind Selection :	C&C Wind, Leeward (5)
Trib. Area : Span :	Length^2/3
Wall Lateral Pressure :	-9.7 psf
Parapet Lateral Pressure :	
RO Lateral Pressure :	4-Ways
Lateral element force multiplier	
Strength :	1.0
Deflection :	0.7
Header:	Box (lateral top, bottom)
Gravity Load at Header:	10 psf
Additional Pt. Load ea. Stud :	220 lbs
Additional Jamb Axial Load :	865 lbs

Back-to-Back Member L/6 Interconnection Spacing per AISI S100 I1.1

Member	Span	Cantilever
Jamb Studs	36.5 in	0.0 in

See AISI S100 I1.1 for Add'nl Requirements

Brace Settings

Component(s)	Members(s)	Flexural Bracing	Axial KyLy	Axial KtLt	Distortional K-Phi(lb-in/in)	Distortional Lm	Interconnection Spacing
Jamb Studs	600S162-54(50), Back-To-Back	Full	60 in	60 in	0	None	12 in
Vertical Header	600S200-43(33), Boxed	Full	N/A	N/A	0	None	N/A
Lat. Top Head	600T125-54(50), Single	Full	N/A	N/A	0	None	N/A
Lat. Bottom Head	600T125-54(50), Single	Full	N/A	N/A	0	None	N/A

Analysis Results

Component(s)	Members(s)	Axial Load (lb)	Max KL/r	Max. Moment (ft-lb)	Max. Shear (lb)	Bottom Reaction (lb)	Top or End Reaction (lb)
Jamb Studs	600S162-54(50), Back-To-Back	1236.9	97	1093.7	228.6	-243.4	-156.4
Vertical Header	600S200-43(33), Boxed	N/A	N/A	325.4	371.9	N/A	371.9
Lat. Top Head	600T125-54(50), Single	N/A	N/A	76.1	87.0	N/A	87.0
Lat. Bottom Head	600T125-54(50), Single	N/A	N/A	17.3	14.9	N/A	14.9

Design Results

Component(s)	Members(s)	Deflection Span	Deflection Parapet	A + M Interaction	V + M Interaction	Web Stiffners	Design OK
Jamb Studs	600S162-54(50), Back-To-Back	L/883	L/0	0.424	0.24	No	Yes
Vertical Header	600S200-43(33), Boxed	L/9265	NA	0.11	0.15	No	Yes
Lat. Top Head	600T125-54(50), Single	L/23630	NA	0.05	0.05	No	Yes
Lat. Bottom Head	600T125-54(50), Single	L/108130	NA	0.01	0.01	No	Yes

Simpson Strong-Tie® Connectors @ Jambs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	-156.41	0.00	By Others & Anchorage Designed by Engineer	NA	NA

R1	-243.41	1499.38	600T125-54 (50) & (2) .157", 3/4" embed SST PDPA/PDPAT to 4000 nw concrete	21.53 %	90.15 %

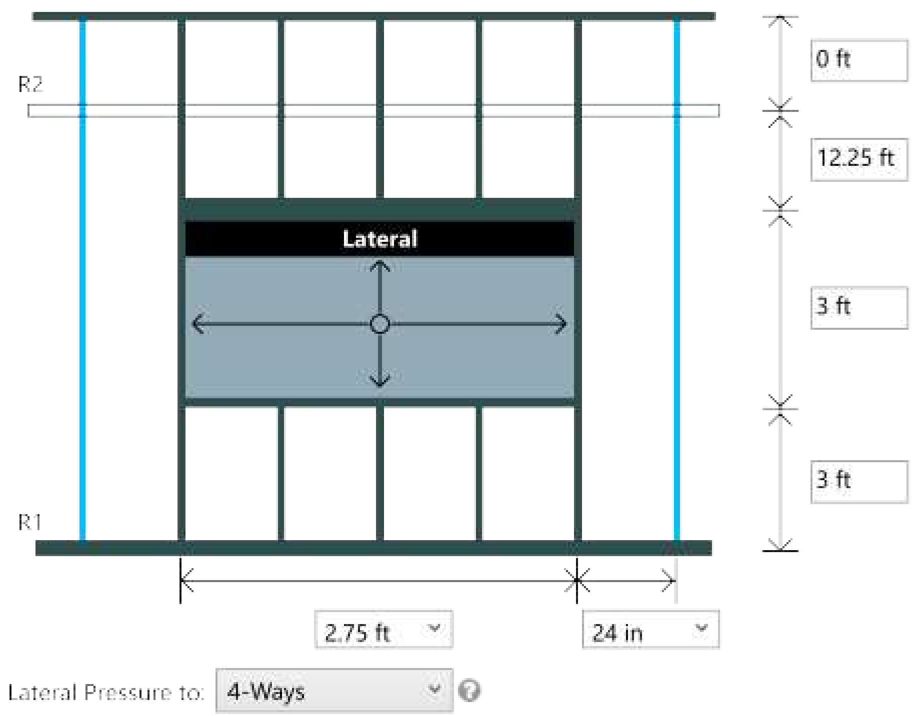
* Reference catalog for connector and anchor requirement notes as well as screw placements requirement

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Jambs

Span/Parapet	Bracing Length(in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min)[1]	LSUBH (Max)[1]	SUBH (Min)[1]	SUBH (Max)[1]	MSUBH (Min)[1]	MSUBH (Max)[1]
Span	Varies	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes:

1) Values in parentheses are stress ratios.
2) Bridging connectors are not designed for back-back, box, or built-up sections.
3) Reference www.strongtie.com for latest load data, important information, and general notes.
4) CFS Designer will not select bridging connectors unless all flexural and axial bracing settings are the same.
5) If the bracing length is larger than the span length, bridging connectors are not designed.



Design Loads

Wind Selection :	C&C Wind, Leeward (5)
Trib. Area : Span :	Length^2/3
Wall Lateral Pressure :	-9.7 psf
Parapet Lateral Pressure :	
RO Lateral Pressure :	4-Ways
Lateral element force multiplier	
Strength :	1.0
Deflection :	0.7
Header:	Single Member
Gravity Load at Header:	10 psf

Brace Settings

Component(s)	Members(s)	Flexural Bracing	Axial KyLy	Axial KtLt	Distortional K-Phi(lb-in/in)	Distortional Lm	Interconnection Spacing
Jamb Studs	600S162-54(50), Single	Full	60 in	60 in	0	None	N/A
Vertical Header	600S162-43(33), Y-Y Axis	Full	N/A	N/A	0	None	N/A
Lateral Header	600S162-43(33), Single	Full	N/A	N/A	0	None	N/A
Sill	600T125-54(50), Single	Full	N/A	N/A	0	None	N/A

Analysis Results

Component(s)	Members(s)	Axial Load (lb)	Max KL/r	Max. Moment (ft-lb)	Max. Shear (lb)	Bottom Reaction (lb)	Top or End Reaction (lb)
Jamb Studs	600S162-54(50), Single	410.9	105	846.6	190.2	-190.2	-128.5
Vertical Header	600S162-43(33), Y-Y Axis	N/A	N/A	115.8	168.4	N/A	168.4
Lateral Header	600S162-43(33), Single	N/A	N/A	64.6	90.9	N/A	90.9
Sill	600T125-54(50), Single	N/A	N/A	22.2	29.2	N/A	29.2

Design Results

Component(s)	Members(s)	Deflection Span	Deflection Parapet	A + M Interaction	V + M Interaction	Web Stiffners	Design OK
Jamb Studs	600S162-54(50), Single	L/557	L/0	0.54	0.37	No	Yes
Vertical Header	600S162-43(33), Y-Y Axis	L/697	NA	0.65	0.65	No	Yes
Lateral Header	600S162-43(33), Single	L/36834	NA	0.06	0.07	No	Yes
Combined Header				0.71	0		
Sill	600T125-54(50), Single	L/104904	NA	0.01	0.01	No	Yes

Simpson Strong-Tie® Connectors @ Jambs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction

R2	-128.53	0.00	By Others & Anchorage Designed by Engineer	NA	NA
R1	-190.21	410.94	600T125-54 (50) & (3) .157", 3/4" embed SST PDPA/PDPAT to 2500 nw concrete	20.44 %	52.84 %

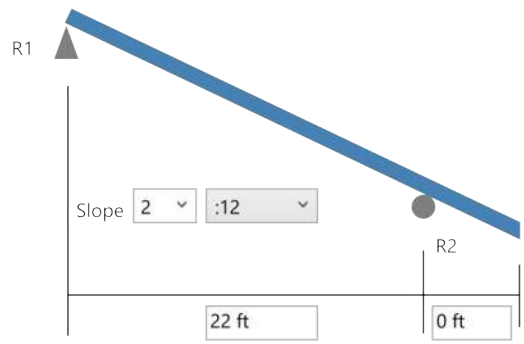
* Reference catalog for connector and anchor requirement notes as well as screw placements requirement

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Jambs

Span/Parapet	Bracing Length(in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min)[1]	LSUBH (Max)[1]	SUBH (Min)[1]	SUBH (Max)[1]	MSUBH (Min)[1]	MSUBH (Max)[1]
Span	Varies	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes:

1) Values in parentheses are stress ratios.
2) Bridging connectors are not designed for back-back, box, or built-up sections.
3) Reference www.strongtie.com for latest load data, important information, and general notes.
4) CFS Designer will not select bridging connectors unless all flexural and axial bracing settings are the same.
5) If the bracing length is larger than the span length, bridging connectors are not designed.



Section : 1000S250-68 (50 ksi) @ 24" o.c. Single C Stud (punched)		
Maxo = 6907.2(ft-lb)	**Va =** 3345.4	**I =** 15.741

Bracing, Interconnection and Distortional Buckling Parameters

	Span	Overhang
Flexural Bracing	96 in	None
Axial Bracing	NA	None
Distortional Buckling, K-phi	0 lb-in/in	0 lb-in/in
Distortional Buckling Bracing, Lm	None	None

Load Cases

	Span (psf)	Overhang (psf)
Dead Load	13	NA
Live Load	20	NA
Snow Load	25	NA
Inward Wind Load	16	NA
Outward Wind Load	-29	NA

Load Combinations

LC Number	Dead	Live	Snow	Inward Wind	Outward Wind
1	1	1	0	0	0
2	1	0	1	0	0
3	1	0	0	0.6	0
4	1	0.75	0	0.45	0
5	1	0	0.75	0.45	0
6	0.6	0	0	0	0.6

Reactions

	Vertical				Horizontal			
	Max Rxn (lb)	Load Comb.	Min Rxn (lb)	Load Comb.	Max Rxn (lb)	Load Comb.	Min Rxn (lb)	Load Comb.
R1	856.45	5	-198.20	6	70.40	3	-127.60	6
R2	865.25	5	-219.47	6	0.00	1	0.00	1

Rafter Flexural and Deflection

Mmax (ft-lb)	Ma (ft-lb)	Mmax/ Ma	Load Comb.	Span Defl	Load Comb.	Overhang Defl	Load Comb.
4759	5271	0.90	5	L/292	5	L/292	5

Rafter Bending and Web Crippling

Support	Load (lb)	Load Comb.	Bearing (in)	Pa (lb)	Pn (lb)	Max Intr.	Load Comb.	Stiffeners Req'd
R1	853.5	5	6.00	1492.4	2611.7	0.30	5	NO
R2	853.5	5	6.00	1492.4	2611.7	0.30	5	NO

Rafter Bending and Shear

Support	Vmax (lb)	Load Comb.	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Load Comb.	Intr. Stiffen	Load Comb.
R1	853	5	1.000	0.26	0.00	0.26	5	N/A	N/A
R2	853	5	1.000	0.26	0.00	0.26	5	N/A	N/A

FCB/MFCB Bypass Framing Fixed-Clip Connector

This product is preferable to similar connectors because of a) easier installation, b) higher loads, c) lower installed cost, or a combination of these features.

The FCB/MFCB clip is an economical, high-performance fixed-clip connector that can be used for a variety of framing applications. It is rated for tension, compression, shear and in-plane loads and offers the designer the flexibility of specifying different screw and anchorage patterns that conform to desired load levels.

Features:

- Rated for tension, compression, shear and in-plane loads
- Provides design flexibility with varying screw and anchorage patterns that achieve different load levels
- Strategically placed stiffeners, embossments and anchor holes maximize connector performance

Material: FCB — 54 mil (16 ga.); MFCB — 68 mil (14 ga.)

Finish: Galvanized (G90)

Installation:

- Use the specified type and number of anchors.
- Use the specified number of #12 self-drilling screws to CFS framing. Note that #10 self-drilling screws can be used per the load tables given on **strongtie.com**.
- For installations to wood framing, see Simpson Strong-Tie® engineering letter L-CF-FIXCLIPW at **strongtie.com**.

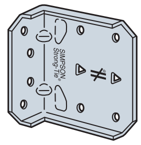
Codes: See p. 13 for Code Reference Key Chart

Ordering Information:

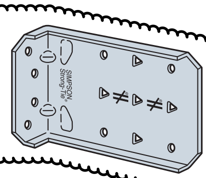
FCB43.5-R25, MFCB43.5-R25, FCB45.5-R25, MFCB45.5-R25, FCB47.5-R25, MFCB47.5-R25, FCB49.5-R25, FCB411.5-R25 contain:

- Box of 25 connectors (screws not included)

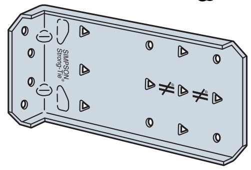
FCB43.5 *(MFCB43.5 similar)* NEW



FCB45.5 *(MFCB45.5 similar)* NEW

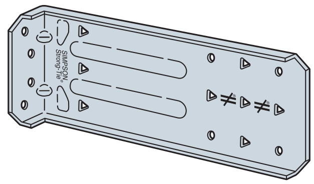


FCB47.5 *(MFCB47.5 similar)* NEW

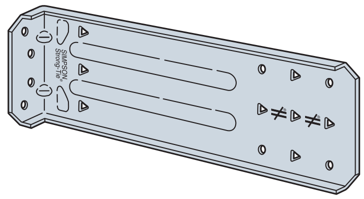


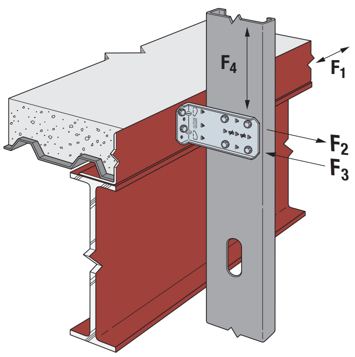
US Patent: 8,555,592

FCB49.5

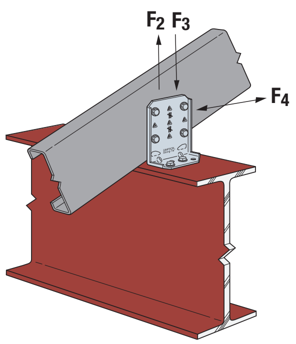


FCB411.5

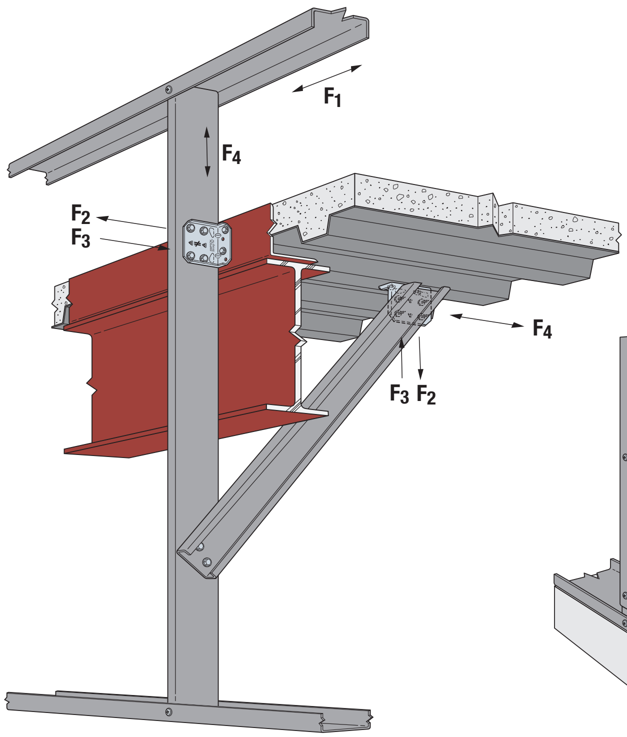




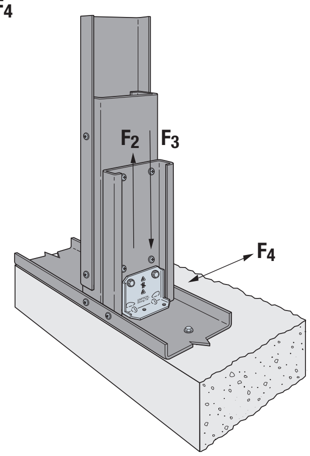
Typical FCB/MFCB Installation at Bypass Framing



Typical FCB/MFCB Installation for Roof Rafters



Typical FCB/MFCB Installation at Sprandrel Studs and Kickers



Typical FCB/MFCB Installation at the Base of a 6" Jamb Stud

Rigid Connectors

SIMPSON Strong-Tie

FCB/MFCB Bypass Framing Fixed-Clip Connector

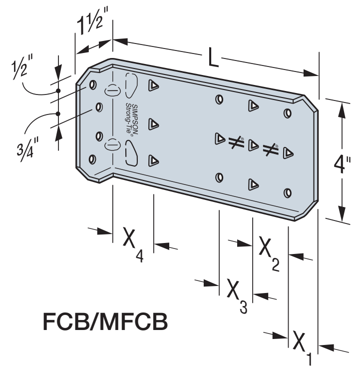
FCB/MFCB Allowable Connector Loads (lb.)

Model No.	Connector Material Thickness mil (ga.)	L (in.)	Min./Max.	No. of #12–14 Self-Drilling Screws	Stud Thickness												Code Ref.
					33 mil (20 ga.)				43 mil (18 ga.)				54 mil (16 ga.)				
					F_1[3,4]	F_2	F_3	F_4	F_1[3,4]	F_2	F_3	F_4	F_1[3,4]	F_2	F_3	F_4	
FCB43.5	54 (16)	3½	Min.	4	140	755	755	755	175	1,105	905	1,055	330	1,250	905	1,235	IBC, FL, LA
			Max.	6	205	1,100	1,130	1,075	260	1,105	1,105	1,350	330	1,250	2,245	1,770	
MFCB43.5	68 (14)	3½	Min.	4	140	755	755	755	220	1,105	1,105	1,055	410	1,530	2,280	1,595	
			Max.	6	205	1,130	1,130	1,075	260	1,265	1,105	1,545	410	1,530	2,630	1,770	
FCB45.5	54 (16)	5½	Min.	4	120	755	755	700	150	1,105	905	875	285	1,105	905	1,100	
			Max.	9	155	1,100	1,260	1,095	195	1,105	1,105	1,380	330	1,105	2,245	1,785	
MFCB45.5	68 (14)	5½	Min.	4	170	755	755	700	220	1,105	1,105	1,030	410	1,530	2,280	1,595	
			Max.	9	170	1,265	1,260	1,695	220	1,265	1,105	2,315	410	1,605	3,205	2,315	
FCB47.5	54 (16)	7½	Min.	4	90	755	755	220	110	1,105	875	330	215	1,105	875	815	
			Max.	12	110	1,100	1,260	705	135	1,105	1,260	1,050	260	1,105	2,245	1,345	
MFCB47.5	68 (14)	7½	Min.	4	165	755	755	415	215	1,105	1,105	540	410	1,580	2,280	1,025	
			Max.	12	165	1,265	1,260	1,345	215	1,265	1,405	1,530	410	1,605	3,350	2,700	
FCB49.5	54 (16)	9½	Min.	4	—	755	755	170	—	1,105	905	255	—	1,105	905	340	
			Max.	12	—	1,100	1,260	750	—	1,105	1,260	1,115	—	1,105	2,245	1,200	
FCB411.5	54 (16)	11½	Min.	4	—	755	755	140	—	1,105	935	205	—	1,105	935	340	
			Max.	12	—	1,100	1,260	795	—	1,105	1,260	860	—	1,105	2,245	860	

1. Min. fastener quantity and load values — fill all round holes; max. fastener quantity and load values — fill all round and triangular holes.
2. Allowable loads are based on clip capacity only and do not consider anchorage. The capacity of the connection system will be the minimum of the tabulated value and the allowable load from the FCB/MFCB Allowable Anchorage Loads table on p. 75.
3. Anchorage to the supporting structure using welds or a minimum of (2) #12–24 self-drilling screws is required.
4. Tabulated F_1 loads are based on assembly tests with the load through the centerline of stud. Tested failure modes were due to screw pullout; therefore compare F_1 against F_p calculated per ASCE 7-16 Chapter 13 with a_p = 1.25 and R_p = 1.0.
5. Tabulated values for 54 mil (16 ga.) CFS framing may be used for 68 mil (14 ga.) and greater steel thickness.

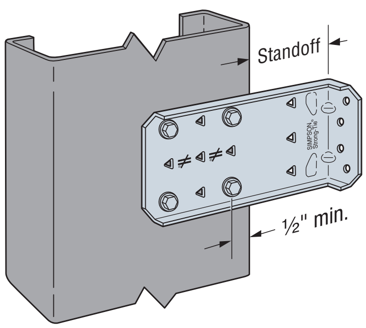
FCB/MFCB Standoff Distances

Model No.	L (in.)	Min./Max.	No. of #12–14 Self-Drilling Screws	Maximum Standoff (in.)
FCB43.5	3½	Min.	4	1
		Max.	6	1
MFCB43.5	3½	Min.	4	1
		Max.	6	1
FCB45.5	5½	Min.	4	1½
		Max.	9	1
MFCB45.5	5½	Min.	4	1½
		Max.	9	1
FCB47.5	7½	Min.	4	3½
		Max.	12	1
MFCB47.5	7½	Min.	4	3½
		Max.	12	1
FCB49.5	9½	Min.	4	5½
		Max.	12	1
FCB411.5	11½	Min.	4	7½
		Max.	12	1

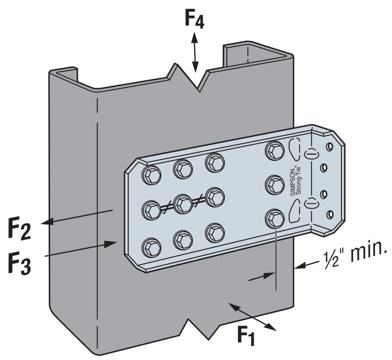


FCB/MFCB

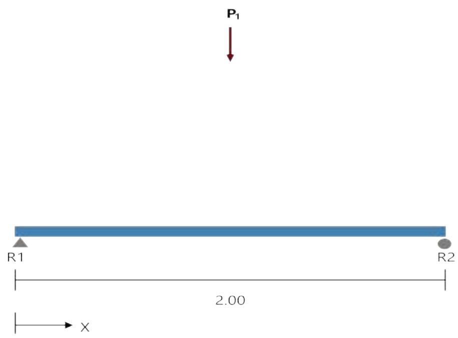
Variable	Dimensions (in.)				
	FCB/MFCB				
	43.5	45.5	47.5	49.5	411.5
X_1	¾	1	1	1	1
X_2	1¼	1¼	1¼	1¼	1¼
X_3	—	1¼	1¼	1¼	1¼
X_4	—	—	1½	1½	1½
L	3½	5½	7½	9½	11½



FCB/MFCB Installation with Min. Fasteners



FCB/MFCB Installation with Max. Fasteners



Section: 600S200-54 (50 ksi) Y-Y Axis C Stud (punched)
Mayo = 499.8 ft-lb **Va =** 3644.3 lb **I =** 0.24 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	NA	Full, N/A	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R1	432.50	1.00	1067.7	0.0	0.20	NO
R2	432.50	1.00	1067.7	0.0	0.20	NO
P1	865.00	1.50	2013.8	432.5	0.74	NO

Point Loads	P1
Load(lb)	865.00
X-Dist.(ft)	1.00

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	0.0(t)	-	0%	KΦ=0.00 lb-in/in Max KL/r = N/A
	Max. Shear, lbs	432.5	3644.3	12%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	432.5	499.8	87%	
	Moment Stability, ft-lbs	432.5	499.8	87%	
	Shear/Moment	0.87	1.00	87%	Shear 432.5, Moment 432.5
	Axial/Moment	0.87	1.00	87%	Axial 0.0(c), Moment 432.5
	Deflection Span, in	0.035	--meets L/692--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R1	0.0	432.5	NA with Y-Y Axis design & NA	0.00 %	0.00 %
R2	0.0	432.5	NA with Y-Y Axis design & NA	0.00 %	0.00 %

* Reference catalog for connector and anchor requirements notes as well as screw placement requirements

Screw Capacities

Table Notes

1. Capacities based on AISI S100 Section E4.
2. When connecting materials of different steel thicknesses or tensile strengths, use the lowest values. Tabulated values assume two sheets of equal thickness are connected.
3. Capacities are based on Allowable Strength Design (ASD) and include safety factor of 3.0.
4. Where multiple fasteners are used, screws are assumed to have a center-to-center spacing of at least 3 times the nominal diameter (d).
5. Screws are assumed to have a center-of-screw to edge-of-steel dimension of at least 1.5 times the nominal diameter (d) of the screw.
6. Pull-out capacity is based on the lesser of pull-out capacity in sheet closest to screw tip or tension strength of screw.
7. Pull-over capacity is based on the lesser of pull-over capacity for sheet closest to screw header or tension strength of screw.
8. Values are for pure shear or tension loads. See AISI Section E4.5 for combined shear and pull-over.
9. Screw Shear (Pss), tension (Pts), diameter, and head diameter are from CFSEI Tech Note (F701-12).
10. Screw shear strength is the average value, and tension strength is the lowest value listed in CFSEI Tech Note (F701-12).
11. Higher values for screw strength (Pss, Pts), may be obtained by specifying screws from a specific manufacturer.

Allowable Screw Connection Capacity (lbs)

Thickness (Mils)	Design Thickness	Fy Yield (ksi)	Fu Tensile (ksi)	#6 Screw (Pss = 643 lbs, Pts = 419 lbs) 0.138" dia, 0.272" Head			#8 Screw (Pss= 1278 lbs, Pts = 586 lbs) 0.164" dia, 0.272" Head			#10 Screw (Pss= 1644 lbs, Pts = 1158 lbs) 0.190" dia, 0.340" Head			#12 Screw (Pss= 2330 lbs, Pts = 2325 lbs) 0.216" dia, 0.340" Head			¼" Screw (Pss= 3048 lbs, Pts = 3201 lbs) 0.250" dia, 0.409" Head		
				Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over
18	0.0188	33	33	44	24	84	48	29	84	52	33	105	55	38	105	60	44	127
27	0.0283	33	33	82	37	127	89	43	127	96	50	159	102	57	159	110	66	191
30	0.0312	33	33	95	40	140	103	48	140	111	55	175	118	63	175	127	73	211
33	0.0346	33	45	151	61	140	164	72	195	177	84	265	188	95	265	203	110	318
43	0.0451	33	45	214	79	140	244	94	195	263	109	345	280	124	345	302	144	415
~~54~~	~~0.0566~~	~~33~~	~~45~~	~~214~~	~~100~~	~~140~~	~~344~~	~~118~~	~~195~~	~~370~~	~~137~~	~~386~~	~~394~~	~~156~~	~~433~~	~~424~~	~~180~~	~~521~~
~~68~~	~~0.0713~~	~~33~~	~~45~~	~~214~~	~~125~~	~~140~~	~~426~~	~~149~~	~~195~~	~~523~~	~~173~~	~~386~~	~~557~~	~~196~~	~~545~~	~~600~~	~~227~~	~~656~~
~~97~~	~~0.1017~~	~~33~~	~~45~~	~~214~~	~~140~~	~~140~~	~~426~~	~~195~~	~~195~~	~~548~~	~~246~~	~~386~~	~~777~~	~~280~~	~~775~~	~~1,016~~	~~324~~	~~936~~
~~118~~	~~0.1242~~	~~33~~	~~45~~	~~214~~	~~140~~	~~140~~	~~426~~	~~195~~	~~195~~	~~548~~	~~301~~	~~386~~	~~777~~	~~342~~	~~775~~	~~1,016~~	~~396~~	~~1,067~~
54	0.0566	50	65	214	140	140	426	171	195	534	198	386	569	225	625	613	261	752
68	0.0713	50	65	214	140	140	426	195	195	548	249	386	777	284	775	866	328	948
97	0.1017	50	65	214	140	140	426	195	195	548	356	386	777	405	775	1,016	468	1,067
118	0.1242	50	65	214	140	140	426	195	195	548	386	386	777	494	775	1,016	572	1,067

SUPREME Allowable Screw Connection Capacity (Pounds Per Screw)

Thickness (mil)	Design Thickness (in)	Fy Yield (ksi)	Fu Tensile (ksi)	#6 Screw (Pss = 643 lbs, Pts = 419 lbs) 0.138" Dia; 0.272" Head			#8 Screw (Pss= 1278 lbs, Pts = 586 lbs) 0.164" Dia; 0.272" Head			#10 Screw (Pss= 1644 lbs, Pts = 1158 lbs) 0.190" Dia; 0.340" Head			#12 Screw (Pss= 2330 lbs, Pts = 2325 lbs) 0.216" Dia; 0.340" Head			¼" Screw (Pss= 3048 lbs, Pts = 3201 lbs) 0.250" Dia; 0.409" Head		
				Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over
D25	0.0155	50	65	111	39	137	111	47	137	111	54	171	-	-	-	-	-	-
D20	0.0188	57	65	142 [1]	48	140	150 [1]	57	166	164 [1]	66	208	109	75	208	-	-	-
30EQD	0.0235	57	65	174 [1]	60	140	184 [1]	71	195	236 [1]	82	260	152	93	260	-	-	-
33EQD	0.0235	57	65	174 [1]	60	140	184 [1]	71	195	236 [1]	82	260	152	93	260	-	-	-
33EQS	0.0295	57	65	171	75	140	187	89	195	201	103	326	214	117	326	231	136	392
43EQS	0.0400	57	65	270	102	140	295	121	195	317	140	386	338	159	442	364	184	532

[1]Values are based on testing using AISI S100 procedures.